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**FRIGID INSERTION METHOD FOR
PIEZOMETER INSTALLATION IN CLAY SOILS**

by

Christopher M. Hudec

A Thesis

**Submitted to the Faculty of Graduate Studies and Research
through the Department of Geological Engineering
in Partial Fulfilment of the Requirements for the
Degree of Master of Applied Science at the
University of Windsor**

**Windsor, Ontario, Canada
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ABSTRACT

The standard method of piezometer installation in clayey soils results in field-measured hydraulic conductivities (K) that are less than the actual values (D'Astous et al., 1989). This difference has been attributed to the smearing of the clayey soils during the drilling process. Smearing of the fractures can therefore lead to an underestimation of the velocity of groundwater and the velocity of chemicals in the groundwater.

The main objective of this investigation was to develop a piezometer insertion technique that would give a more representative field-determined hydraulic conductivity. The investigation consisted of both laboratory and field tests.

In the laboratory test, laboratory-fractured remolded clay columns were drilled at both room temperature and in a frozen state. The columns were then tested for hydraulic conductivity using a constant head permeability test. This test indicated that drilling the clay when frozen reduced smearing of the clay. The geometric mean of the K values obtained from tests where the clay was drilled at room temperature was 3.8×10^{-3} m/sec. The geometric mean of the K values where the clay was frozen before drilling was 2.6×10^{-4} m/sec. The increase in K is attributed to reduced smearing of the fracture.

In the field tests, 16 piezometers were installed at a field site characterized by fractured clayey soils using the Overcored Shelby-Hole Method (OSHM) of D'Astous et al. (1989). D'Astous et al. (1989) showed that the OSHM method gave field-measured K values that exceeded the field-measured K values for conventionally installed piezometers by one to three orders of magnitude.

Another 16 piezometers were installed using a technique developed in this study

called the Frigid Insertion Method (Frigin). The Frigin method is a modification of the OSHM in which the clayey soil is frozen in situ as part of the OSHM approach.

The K values in all of the piezometers were determined three times using the Hvorslev (1951) method. The Frigin piezometers yielded higher K values than the OSHM piezometers. The geometric means of the K values were 2.0×10^{-6} m/sec and 4.8×10^{-6} m/sec for the OSHM and Frigin methods, respectively (including outliers).

DEDICATION

The motivation provided by my parents was greatly appreciated. When I graduated from elementary school, my father told me I had to go to middle school. When I complained to my mother, she said the same thing. They also said I had to go to high school after I finished middle school, and likewise university after high school. After I got my B.A.Sc., my father recommended that I get a M.A.Sc.. Since I was on a roll I decided to humour him. Honestly, I know I would not have the education I now have if it was not for them.

Many thanks to Anne and Keith Nelligan. I am grateful for their time, insight, understanding, and a place to go when I needed to escape this thesis. I'm not sure I would have remained (?) sane if it wasn't for them.

Thanks to all the people who helped me and were not mentioned.

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1.0 INTRODUCTION

1.1 Background

Thick clay deposits have long been thought of as good media for the disposal of wastes. Due to the extremely slow transmission of water through massive clays, the transport of leachate and other liquid wastes through clays is also perceived to be very slow (Freeze and Cherry, 1979; Prudic, 1982). This perception is based on published hydraulic conductivities of unweathered clay till, which range between 10^{-8} and 10^{-10} cm/s (Freeze and Cherry, 1979). At this range of hydraulic conductivities and the typical lateral hydraulic gradients in these environments, it would take contaminated water at least 300 years to flow 1 m.

This is not the case, however, for near-surface clayey soils. There are many documented instances where waste has leaked out of hazardous waste and landfill sites (Kueper et al., 1992; McKay and Fredricia, 1989; Hendry, 1993; Harrison et al., 1992). In all of these instances, contaminants have leaked through clay liners and natural deposits into underlying aquifers.

Numerous studies of tritium distributions indicate "young" (<40 years) ground water at depths of 3 m or more in clay plains (Ruland et al. 1991; D'Astous et al. 1989; Keller et al. 1986). Young ground water at these depths indicates relatively high hydraulic conductivities. The ground water most likely reached these depths through fracture networks in the surficially weathered clay Rowe and Booker (1991) Rudolph et al. (1991). These fracture networks increase the bulk hydraulic conductivity of the soil, which accounts for both flow through fractures and the intervening massive clay blocks.

1.2 Objectives

The main objective of this research project has been the development of a modification to standard piezometer installation procedures that would preserve the integrity of the fractures that exist in near-surface clayey soils.

The most common method for piezometer installation in unconsolidated soils involves augers, either hollow stem or solid, for drilling the hole. The rotation of the auger flights smears the clay along the walls of the borehole, at least partially sealing fractures that the borehole intersects. When these monitoring wells are used to assess the hydraulic conductivity (K), they yield a low K indicative of massive clay (D'Astous et al., 1989).

More advanced techniques of monitoring well installation employ one or more Shelby tubes and avoid rotation to remove the soil from the area where the screened portion of the piezometer resides. These methods reduce the smearing caused by the auger rotation, but they still allow some smearing to occur. This thesis addresses these problems by the development of a piezometer installation technique that further reduces smearing.

1.3 Scope

The main technique employed in this investigation is the freezing of clay soil prior to piezometer insertion. The aim of the freezing is to reduce smearing. Using this technique, the hydraulic conductivity values obtained during field hydraulic conductivity tests should more closely reflect the actual field conditions.

1.4 Structure of Thesis

The thesis consists of eight chapters. The first chapter gives the objective and structure. The second chapter includes the literature review and discusses theoretical considerations including the mechanical and thermal properties of soils. The third chapter describes the study area while Chapter 4 outlines the laboratory and field methods. Chapter 5 presents the results and discussions, and Chapter 6 gives the conclusions and recommendations. Chapters 7 and 8 give the references and appendices, respectively.

2.0 THEORETICAL CONSIDERATIONS

2.1 Fractured Clay Soils

The growing concern over the frequency and depth of vertical fractures in clay plains and the inability of the soils to contain waste are the stimulus for this project. Recognizing the impact of fractures on the bulk hydraulic conductivity of clayey soils is critical in estimating the lateral movement of groundwater and chemicals in the groundwater.

2.1.1 Location of Fractured Clay Soils

The uppermost 2 to 3 m of surficial clay tills in clay plains are fractured and weathered (Mase et al., 1990; Morris et al., 1992; Keller et al., 1986; Rowe and Booker, 1990; Morris et al., 1992). Weathered fractures and root holes extend to depths of 4.5 m. Weathered clay has been found at depths of up to 25 m in southern Alberta (Hendry et al., 1986). Although not usually obvious, fractures have been detected at depths of 12 m on the basis of tritium profiles at a hazardous waste disposal facility in Sarnia Ontario (McKay and Cherry, 1992). Models predict possible fracture depths of up to 15 m (Mase et al., 1990).

Fracture spacing in clayey soils steadily decreases with depth. Fracture spacings of one fracture every 1 cm near the surface to one fracture every 50 cm to 2 m at 4.5 m depth were observed by Ruland et al. (1991), McKay (1991), McKay and Cherry (1992), and others (Figure 1).

Fractures dip vertically or near vertically with no or very slight preferred strike directions in the clay plain of southwestern Ontario (McKay et al., 1993). Near

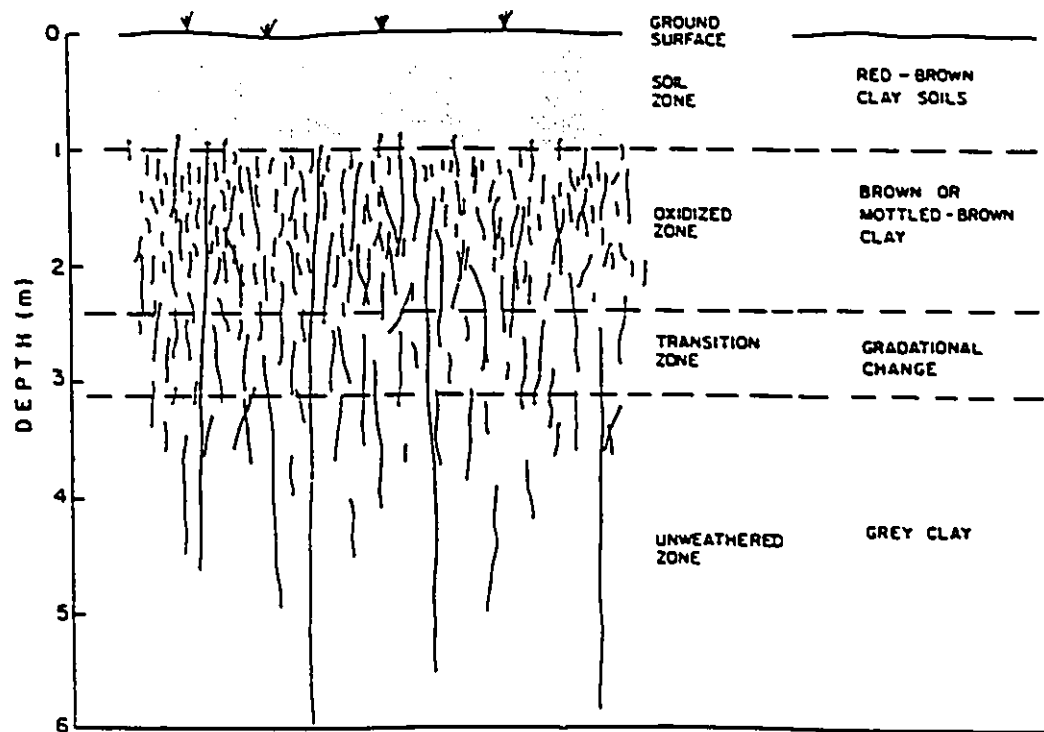


Figure 1. The weathered, transition and unweathered zones commonly observed in clay deposits in southern Ontario (adapted from Ruland et al., 1991)

Smithville Ontario, fractures in the lowermost 2 m of a 6 m deep clay till have strike directions that parallel fractures in the bedrock beneath the clay (Willis et al., 1992). The fractures in the till usually lie directly above the fractures in the bedrock. It seems in this case that the fractures in the bedrock propagated upwards into the clay. The fractures in this till progressively lose their preferred strike directions toward the surface of the clay till, where there are no preferred strike directions.

2.1.2 How Fractures Formed

Fracturing occurs in several ways. Desiccation cracks and associated weathering, "probably formed during the relatively warm Altithermal climatic period that existed between about 9000 and 3000 years ago" (Farvolden and Cherry, 1988; Quigley and Ogunbadjo, 1973). Vorauer et al. (1986) proposed that fault movement can cause fractures in clay. In the same paper, they also postulated that the fractures were a fatigue phenomenon induced by semi-diurnal rock tides caused by the gravitational attraction of the moon (Vorauer et al., 1986).

Root systems also produce macropores in the clay through which water can flow readily. These imperfections in the weathered zone may increase the hydraulic conductivities of the clay by two orders of magnitude to between 10^{-6} and 10^{-8} cm/sec (Sharp, 1984).

It has been shown that fractures occur in clay tills that have not undergone physical weathering (Keller et al., 1985). Fractures in these clays are surmised to have occurred due to the release of the confining pressure of glaciers, crustal rebound after the glaciers receded, and/or tectonic movement.

The process by which the fractures form in the weathered zone is described by Morris et al. (1992). First, moisture is removed through evaporation. This causes desiccation cracks. These cracks allow water to drain deeper into the clay. The water carries dissolved minerals that react with and are deposited on the walls of the fracture, changing the nature of the clay. Frost action and freezing in the winter further deepen and expand the fractures allowing more free water to penetrate deeper into the clay matrix. The fracturing process progresses downwards until a depth is reached where there is always enough water in the massive clay to prevent desiccation cracks and where the ground never freezes.

2.2 Hydraulic Implications of Fractured Clay Soils

The effect of fractures and macropores is rarely observed in field tests of hydraulic conductivity when employing conventional drilling methods to install monitoring wells. It is important to obtain accurate estimates of the hydraulic conductivity (K) because K is used to predict the rate of groundwater flow. If the groundwater flow rate estimate is incorrect, then any predictions or evaluations based on it will also be incorrect.

The regulatory agencies generally require field tests for K (Sklash, personal communication, 1991). Estimates of K derived from isotopic data from wells in shallow weathered clay indicate recent recharge of water, usually much more recent than K values from field tests imply. This difference can be attributed to three major factors: 1) Most of the fractures are vertical or nearly so, and since most wells are also vertical, the probability of intersecting the fractures is small. 2) The frequency of fractures decreases with depth. 3) The drilling technique used to install the test piezometers may smear fractures (Ruland

et al., 1991).

The conventional installation method for piezometers is the augered borehole method. This method involves the placing of the piezometer into a borehole advanced using solid stem or hollow stem augers. The main problem with this technique is that it causes smearing of the clay in the vicinity of the borehole as the auger rotates. This smearing, which affects between 0.1 cm and 0.3 cm of the clay past the auger blades (D'Astous et al. 1989), is sufficient to close fractures transected by the well. Once the fractures are closed, the weathered zone behaves much the same as fresh till resulting in erroneously low hydraulic conductivity values in field K tests.

2.2.1 Permeability / Hydraulic Conductivity of Clayey Soils

D'Astous et al. (1989) conducted an investigation of fracture effects in weathered clay near Sarnia, Ontario. They used several piezometer installation techniques designed to reduce smearing, test pits, and large-diameter (1.5 m) wells in which the smearing was manually removed, to estimate the K value in the weathered zone. In the tests pits and large diameter wells where the smeared zone was removed, K values were on the order of 10^{-6} cm/s. They concluded that these values are representative of the bulk K of the weathered zone which includes the interconnected fractures. The piezometers that were installed using the smearing reduction techniques gave K values higher than wells installed using conventional methods, but less than the large-diameter wells where the smeared clay was manually removed.

The range of hydraulic conductivities derived from tests on conventionally installed piezometers are from 10^{-7} to 10^{-9} cm/s (Richards et al., 1984; Desaulniers,

1986; Harding, 1986; D'Astous et al., 1989). This range of values is up to three orders of magnitude slower than that obtained from modified techniques.

Two techniques proposed by D'Astous et al. (1989) that would increase the chances of obtaining more representative hydraulic conductivity values are (1) increasing the well diameter and (2) installing the boreholes at an angle. Both of these techniques would increase the chances of intersecting fractures but there are problems with both these techniques. Some of the problems with increasing the diameter of the borehole are (1) slightly higher material and installation costs, (2) the volume of water that must be bailed or added for single well testing if a large- diameter well is installed, (3) increased recovery time after installation, (4) more purge water when sampling, and (5) increased effort in piezometer completion. The drawbacks of installing angled boreholes are (1) a slight increase in installation costs, (2) more difficult sampling, (3) more difficult calculation of water height in the piezometer, (4) not all rigs can easily drill angled holes, (5) the need to determine dominant strike, and (6) increased effort for completion and sealing of the piezometer.

2.2.2 Implications

If weathered clay till has a higher actual hydraulic conductivity value than estimated from tests on conventionally installed piezometers, the actual groundwater velocity in the shallow subsurface will be greater than estimated. This would affect the calculation of migration of pesticides and fertilizers in agricultural situations and the lateral transport of contaminants from landfills and hazardous waste sites, among others.

2.3 Theoretical Considerations Concerning The Freezing of Soil

This section focusses on how soils freeze when subjected to temperatures below 0°C. Most work done in this area is on the formation of ice lenses and the associated frost heave. Frost heave damages roads and buildings. Ice lenses can increase the hydraulic conductivity of near-surface clays due to physical weathering.

Freezing is sometimes used to facilitate excavation of tunnels and exploratory shafts where soil conditions, location of the excavation (depth) and groundwater problems warrant the cost of freezing (Legget and Hatheway, 1988). Freezing has also been used to prevent soil movement in small scale landslides associated with construction (Gordon, 1937).

In natural situations, soil freezes slowly. When the air temperature is below freezing, "a thermal gradient is established in which the 0°C point is below the ground surface" (Sowers and Sowers, 1970). This surface is called the "frost line"; it roughly follows the surface topography. The depth of the frost line below the ground surface depends on the temperature and duration of below freezing temperatures.

When soil freezes slowly, ice lenses can form in the soil. Water freezes at 0°C in large pores in the soil, but lower temperatures are required to freeze the water in small pores. The depressed freezing point is caused by the high capillary tension in the small pores. Therefore, ice will form first in the large pores. Water then migrates toward the ice in the large pores and freezes. This increases the size of the intergranular ice, forming an ice lens. Water propagates toward the ice until there is no more free water in the soil or the temperature rises above freezing (Craig, 1978). The formation of an ice lens

fractures the soil around the lens, increasing the bulk K of the soil.

Ice lenses do not form when the soil is frozen quickly. The zone of soil in which the pore water is unfrozen but below 0°C is small. Therefore there is not enough time for the water to migrate, so the lenses do not form (Sowers and Sowers, 1970).

2.4 Mechanism For Heat Transfer

Heat transfer occurs in nature continuously. Heat is always moving from areas of higher temperature to lower temperature. The mechanisms through which this occurs are radiation, convection, and conduction (Incropera and De Witt, 1990).

Convection and conduction are the important mechanisms for this investigation. Conduction is the transfer of heat energy from more energetic, or hotter particles, to less energetic, or cooler particles. The energy is transferred through the vibration of particles against one another. The particles do not actually move from place to place. That is conduction of energy (heat). Conduction occurs primarily in solids and stationary liquids because the particles do not move around with respect to one another (Incropera and De Witt, 1990).

Convection, on the other hand, is a type of molecular kinetic energy. It occurs only in liquids, gases and plasma. An energetic (or hot) molecule or atom transports its energy by moving from one place to another.

For convection and conduction to work, there must be a driving force. That force is called the thermal gradient. A thermal gradient is the difference in temperature between two points over the distance between them. The larger the magnitude of the thermal gradient, the larger is the driving force for heat transfer.

The rate at which energy is withdrawn (in the form of heat) from a cylindrical body is given by Equation 1:

$$q = \frac{2\pi Lk(T_1 - T_2)}{\ln\left(\frac{r_2}{r_1}\right)} \quad (1)$$

where: 'q' is the heat rate, 'L' is the length of the cylinder, 'r₁' and 'r₂' are the radial distances from the centreline of the cylinder, 'T₁' and 'T₂' are the temperatures at the radial distances 'r₁' and 'r₂', and 'k' is the thermal conductivity. This is the one-dimensional, steady-state solution to the heat equation for a cylinder with no generation (Incropera and De Witt, 1990).

Equation 1 gives the heat rate, or the amount of energy absorbed per unit time (in watts). For any given set of conditions (temperature differential and radial distance), the material that the cylinder consists of governs the heat rate (Incropera and De Witt, 1990). In other words, the thermal conductivity gives the 'insulating' properties of the material, which controls the loss or gain of heat energy.

Changing the radial distances increases or decreases the thickness of the 'insulating' material but may not change the heat rate. A critical radius is reached where the surface area of the object can radiate heat faster than it is transferred through the object. At that radius, the heat rate becomes constant. It should also be apparent that if the material is a perfect conductor of heat, the thickness of the material would have no bearing on the amount of heat transmitted. The thermal conductivity of the material would go to infinity and the material would be a superconductor of heat.

2.4.1. Freezing Clay Surrounding a Shelby Hole

In this project, the object being cooled is not a cylinder, but the clay surrounding a Shelby hole. Since the hole has the dimensions of a cylinder, the equation used to calculate the heat rate is the equation for a cylinder. Other shapes (plane walls, spheres, etc.) are described by other equations that change as the surface area (radiating surface) of the object changes. In this case, if the cooling were to go on indefinitely, the area being cooled would change from a near cylinder to a spheroid to a sphere. To prevent this from occurring, a control volume needs to be established that fixes the radial distances being considered (Figure 2). Establishing a control volume changes the equation from steady-state to unsteady-state, which requires integration to determine the heat rate at defined parameters.

Once the amount of energy needed to freeze the clay is calculated (from Equation 1), the theoretical amount of material (in this project liquid nitrogen) needed to remove the heat can be determined. Equation 2 can be used to determine the amount:

$$m = \frac{q}{h_{fg}} \quad (2)$$

where ' h_{fg} ' is the enthalpy for the material and ' q ' is the heat rate calculated using Equation 1 (Incropera and De Witt, 1990).

2.4.2. Transient Temperature Distribution

The time to freeze the clay to the required radial distance can be obtained by using Heisler charts. These are graphical representations of *approximate* relations for

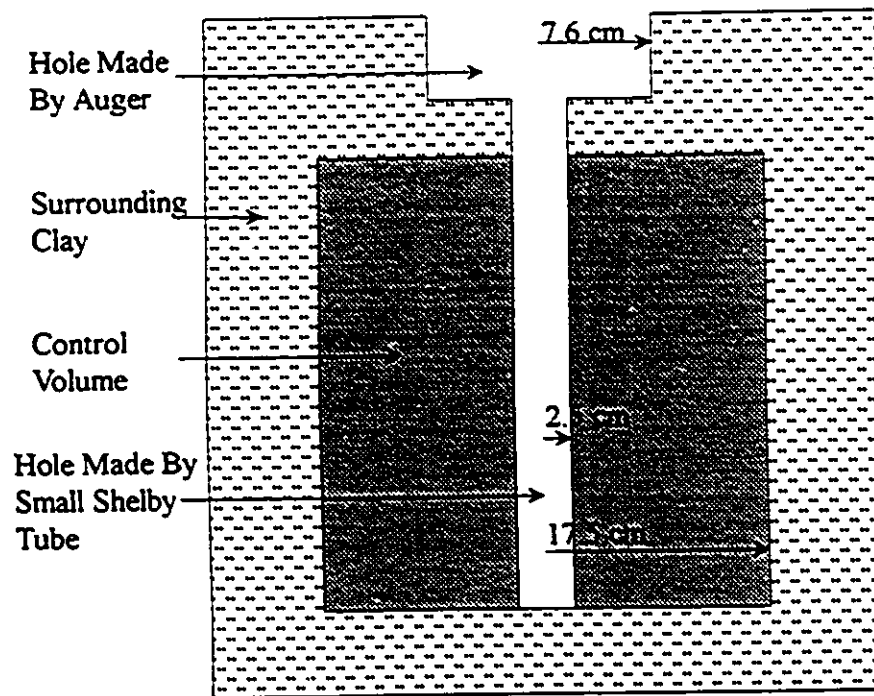


Figure 2. Control volume around Shelby hole.

transient temperature distribution (Heisler, 1947). There are different charts for plane walls, infinite cylinders and spheres. The Heisler chart for an infinite cylinder is shown in Figure 3.

The procedure for obtaining the time is straightforward. First, the Biot number 'Bi' for the cylinder is calculated using Equation 3:

$$Bi = \frac{hr_o}{k} \quad (3)$$

where: 'h' is the convection heat transfer coefficient, 'r_o' is the radius, and 'k' is the thermal conductivity. The Biot number provides a measure of the temperature drop in the solid relative to the temperature difference between the solid's surface and the fluid cooling the solid (Incropera and De Witt, 1990). Because the control volume chosen for this project does not include the liquid nitrogen, no convection takes place. The liquid nitrogen and clay interface, while being a convective surface, is not treated as one. The liquid nitrogen is being used only as a heat sink, to keep the borehole walls at a consistently cold temperature. The cylinder wall is subjected to a sudden change in surface temperature to 77 K (Weast, 1973), and the Biot number becomes zero because 'h' is zero.

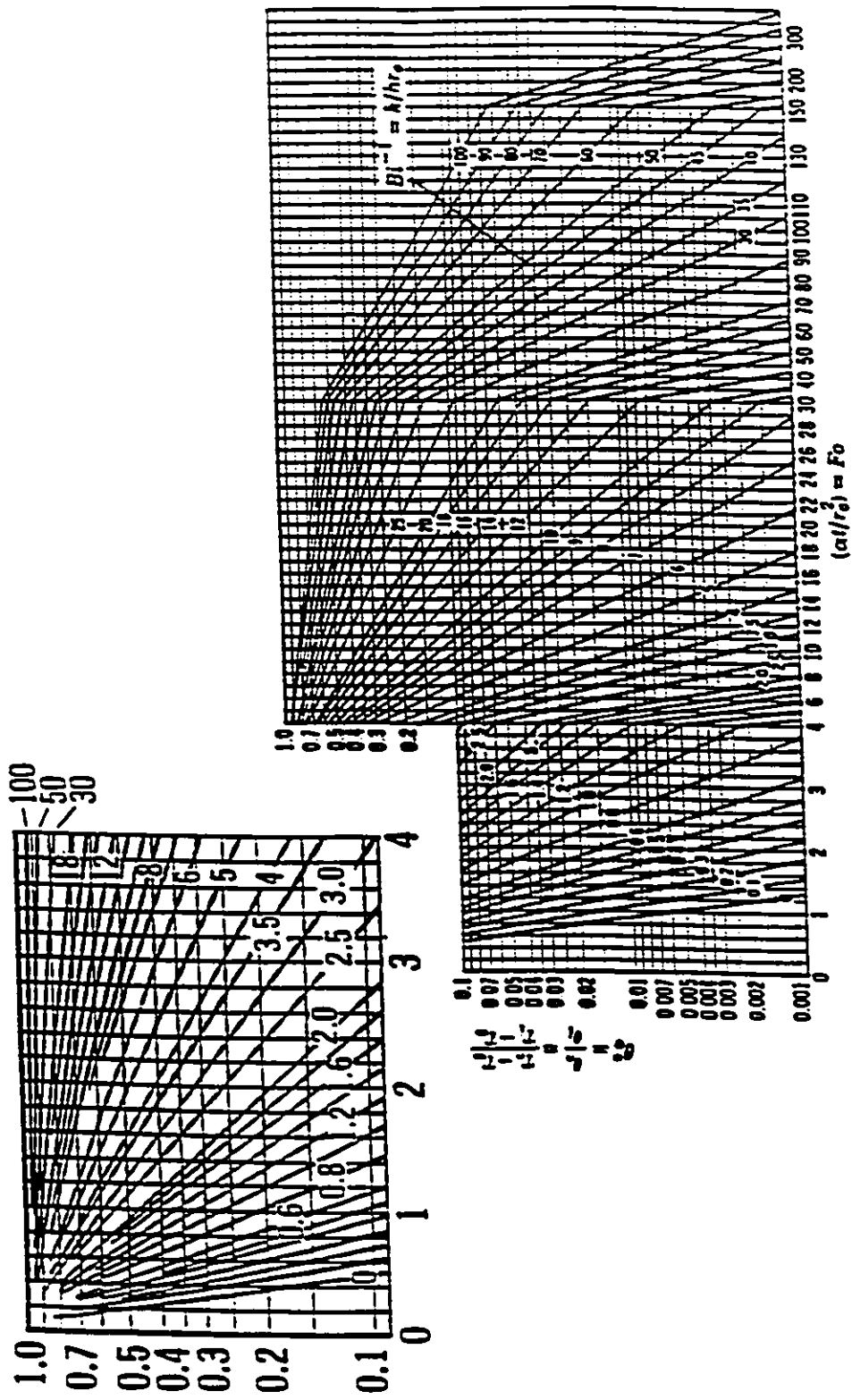


Figure 3. Heisler chart showing centerline temperature as a function of time for an infinite cylinder of radius r_o (from Incropera and De Witt, 1990).

The ratio of temperature differences based on the control volume is calculated using Equation 4:

$$\theta_o = \frac{T_o - T_s}{T_i - T_s} \quad (4)$$

where: ' T_o ' is the temperature at ' r_o ', ' T_i ' is the initial (ambient) temperature of the clay, and ' T_s ' is the temperature at ' r_i '.

The values obtained from Equations 3 and 4 are used to obtain a value from the Heisler charts for the Fourier number 'Fo'. The Fourier number is then used along with Equation 5:

$$Fo = \frac{\alpha t}{r_o^2} \quad (5)$$

to obtain the time 't', where 'alpha' is the thermal diffusivity and ' r_o ' is the radial distance.

Using this procedure, the calculated time to push the 253 K (-20°C) point 17.5 cm radially from the centre of the borehole is approximately 13 minutes. The time and energy needed for the water to change state from a liquid to a solid was not taken into account in the calculations. Including the phase change would have made the calculations very complicated and the increase in accuracy would be small. Overcompensating with the time and energy needed to freeze the clay is much easier and leaves a greater margin for error in the field.

2.4.3. Effect of Freezing on Clay

Freezing the clay prior to drilling removes energy from the water and clay and strengthens the bonds that hold the clay together (Van Vlack, 1985). The stronger bonds are able to absorb more energy than weak ones and later release it without failure. If the bonds break, however, they do so without forming new bonds. Because the water is frozen, the molecules cannot move around to accommodate the applied stresses. With liquid water, the water molecules make and break bonds with each other and the clay easily.

2.5 Mechanics Of Smearing

The mechanical properties of a soil are dependent upon the state of stress in the soil mass and the presence of water in the soil (Sowers and Sowers, 1970). In clays, the amount of preconsolidation affects the present strength of the clay. The amount of water in the clay also affects its strength. Capillary tension and adsorption of water on to clay particles have a great effect on the physical properties.

2.5.1 Overconsolidation

Overconsolidation occurs in clays when the clay has been preconsolidated and the stress is then removed. The clays in southwestern Ontario are overconsolidated because of the overburden pressure exerted by the overlying ice sheet during the last ice age. Since the clay particles did not return to their original positions, the resulting reduction in void ratio (or interparticle spacing) was permanent (Sowers and Sowers, 1970). The reduction in the void ratio also results in a reduction in moisture content. Overconsolidated clays have higher initial strengths than do unconsolidated clays as a

result of the preconsolidation (Diaz-Rodriguez et al., 1992).

2.5.2. Moisture Content

Moisture content has a significant effect on the strength of a clay. The small voids in the clay enable capillary tension to support loads. The void spaces form irregular capillary tubes. In soils that are not saturated with water, a meniscus forms at the air - water interface. The radius of curvature of the meniscus, which is inversely related to the radius of the capillary tube, determines the strength of the tensile forces holding the clay particles together. The smaller the radius of the capillary tubes, the greater the capillary tension (Das, 1985). Strong capillary tension forces increase the strength of the clay.

Clay particles are closely spaced. If a sample of clay is disturbed in a way that will increase its volume, it will resist the applied forces in part by capillary tension (Sowers and Sowers, 1970). The increase in volume of the clay reduces the pressure in the inside of the clay mass. To compensate, some of the water evaporates to form low pressure pockets of gaseous water. The gaseous water "bubbles" cause an interface to form identical to the air - water interface described above (which is more accurately described as a gas - liquid interface). The induced capillary tension increases the strength of the clay (Sowers and Sowers, 1970).

2.6 Smearing

Smearing is caused by failure of the clay in shear. The failure is caused by clay particles sliding or rotating against one another in response to an induced load. The strength of clays is dependent on how the load is applied (Sowers and Sowers, 1970) and on the direction of principal stress (Craig, 1978). Applying the load slowly allows water

to drain out of the pore spaces. The triaxial test in which pore water is allowed to flow out of the clay, is called a drained test (Craig, 1978). This allows the load to be transferred to the soil particles. As the water drains out of the clay, the interparticle spacing decreases. The closer the interparticle spacing, the stronger the clay is and the more resistant it is to failure by shear.

2.6.1 Clay/Water Interaction

In clay minerals, there is an interaction between the solid and water in which the water molecules bind themselves to the clay particle surface (Sowers and Sowers, 1970). This is adsorption of water on to clay. Water is a polar molecule having a positive side and a negative side due to the way the two hydrogen atoms are attached to the oxygen atom. Clay minerals also have charges. The faces of clay particles are negatively charged due to the clay minerals crystal structure, while the edges are positively charged. Because water is dipolar, the opposite charges of the clay face and the oxygen side of the water molecule attract each other, while the edges of the clay particle attract the hydrogen of the water molecule. The water molecules "trapped" by the charge on the clay particles are said to be adsorbed on to the clay (Craig, 1978). Any excess water then acts to lubricate the clay particles, enabling them to move over each other more readily. This gives the clay a lower strength.

After consolidation, the clay minerals are closer together. "The closer spacing increases the particle attraction and reduces potential movement between particles" (Sowers and Sowers, 1970). The interparticle attraction is due to the different charges at the edges and the faces of the clay particles. This attraction increases the strength of the

clay (Lamb, 1951). The reduction in water content also increases the capillary tension in the clay, further increasing strength. If a load is applied quickly, however, the water cannot drain from the clay, and the clay does not consolidate.

The strength of the clay in this case is the undrained strength. The water in the pore spaces supports the load as it is induced and if the load is increased rapidly past the shear strength of the clay, it fails. Hydrostatic pressure effects emerge in the clay because the load is supported by the water in the pores. If the stress in all directions is equal, failure will not occur under any pressure. If a certain differential stress is achieved between any two stress directions, the clay will fail (Frydman and Talesnick, 1992).

2.6.2 Drilling Effects On Clay

In auger drilling, the auger blades cut the clay and carry the cuttings upward. The cuts are failures of the clay in shear. This failure leads to what is commonly called smearing. The smearing is not extensive because there is an unequal distribution of the stresses over the shear surface (D'Astous et al., 1989). Shear diminishes away from the plane of failure (Bergado et al., 1991).

Another technique used to "drill" is using a Shelby tube. A Shelby tube is a thin walled metal tube that is sharpened around its circumference at one end. Figure 4 is a schematic of a standard 3" Shelby tube and Figure 5 is a photograph of the sharpened end of a 3" Shelby tube used in this project. The Shelby tube is hydraulically pressed into the soil. This tube is normally used to take undisturbed soil samples (Das, 1985). Since the thin wall of a Shelby tube minimizes disturbance of the clay, it does not smear the clay as much as auger drilling.

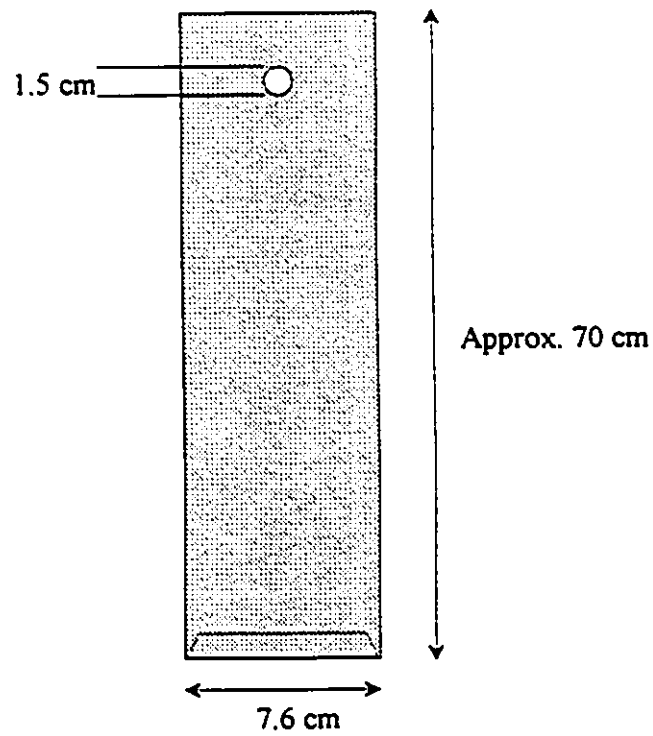


Figure 4. Standard 3" Shelby tube schematic.

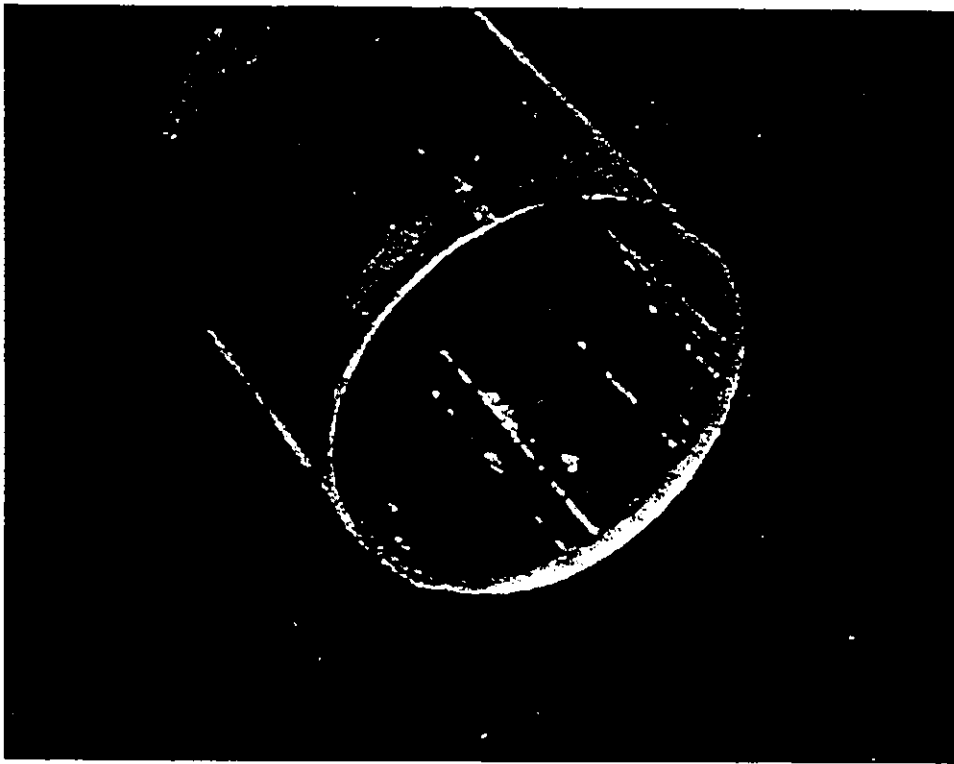


Figure 5. Photograph of 3" Shelby tube showing inside bevel.

When a Shelby tube is used to make a hole, little shear is imposed on the clay. Auger drilling subjects the clay to an intermittent shear force because of the many flights of augers and the rotating blades brushing against the borehole wall. This increases the extent of the shear zone. A Shelby tube reduces the amount of time that the clay is subjected to the shearing force, so its smeared zone is smaller.

In boreholes where the smeared zone was removed manually, D'Astous et al. (1989) observed that the smeared zone ranges from 1 to 3 mm away from the borehole wall. Beyond 3 mm from the borehole wall the clay behaves elastically, and returns to its original state after the load is removed. In the first 3 mm from the borehole wall the clay behaves plastically in response to a load and permanent deformation results. There is not a distinct separation between these two behaviours but a gradual change from one to the other. This change is called "progressive failure" (Sowers and Sowers, 1970). In order to reduce or eliminate the progressive failure, or smeared zone, the clay must behave as a rigid unit. Freezing the clay prior to drilling should provide the necessary rigidity.

2.7 Single Well Response Testing

In the introduction to Hvorslev's paper 'Time Lag and Soil Permeability in Ground-Water Observations' states that the hydrostatic pressure in a borehole or pressure measuring instrument such as a piezometer is seldom equal to the in situ pore water pressure at the time of augering or installation of the device (Hvorslev, 1951). It is after the hole or device is in place that a flow of water takes place to equalize the pressure in the instrument with the in situ hydrostatic pressure.

Hvorslev designated the period required to equalize the pressure differential as the

"time lag". The magnitude of the time lag for a particular borehole, piezometer or other pressure measuring device is dependent on the geometry of the installation and the hydraulic conductivity of the soil surrounding the instrument.

Since it is possible to alter the level of water in the well casing or piezometer after it has reached the same hydrostatic pressure of the surrounding soil, and the dimensions of the casing or piezometer is known, then a mechanism is available to assess the in situ hydraulic conductivity of the soil surrounding the device. This type of in situ testing of hydraulic conductivity is known as "single well response testing".

If 'H' is the static piezometric head measured in an observation well and this head is 'instantaneously' lowered to a piezometric head of 'h', then Hvorslev noted that the flow 'Q' into the well screen at time 't' is proportional to the hydraulic conductivity 'K' of the soil surrounding the screen and the unrecovered head difference 'H - h' (Equation 6):

$$Q(t) = \pi r^2 \frac{dh}{dt} = F_s K (H - h) \quad (6)$$

where: 'r' is the diameter of the observation well and 'F_s' is a shape factor associated with the geometry of the observation inlet. Equation 6 similarly applies when a static piezometric head measured in the observation well, 'h', is raised 'instantaneously' to piezometric head 'H'.

In both instances, it is apparant that Q(t) will decrease at a decreasing rate with increasing time, since the head differential driving the system decreases in magnitude with increasing time. Hvorslev defined the "basic time lag" (T_o) (Equation 7) as:

$$T_o = \frac{\pi r^2}{F_s K} \quad (7)$$

where: ' T_o ' is the time that would be required for the equalization of the initial head difference if the initial rate of flow to the well was constant and (Equation 8):

$$T_o = \frac{V}{Q_o} \quad (8)$$

where: ' V ' is the volume of water initially removed or added to the well. Freeze and Cherry (1979) describe in detail a simple method to determine ' T_o ' from the piezometer recovery data.

Hvorslev (1951) evaluated the shape factor ' F ' for a variety of piezometer arrangements. The arrangement applicable to most installations is a piezometer of screen length (L) and radius (R) such that L/R is greater than 8. In terms of ' T_o ', the hydraulic conductivity of the soil surrounding the well screen is calculated by (Equation 9):

$$K = \frac{r^2 \ln\left(\frac{L}{R}\right)}{2LT_o} \quad (9)$$

3.0 STUDY AREA

3.1 Location and Access

The study area is located in north central Essex County, Ontario, in the Township of Maidstone, approximately 20 km east of the City of Windsor (Figure 6). The field study was conducted on the Eugene Whelan Experimental Farm, a substation of the Harrow Research Station, owned and operated by Agriculture Canada. The property is located on the southwest corner of Essex County Road No. 46 and Maidstone Township Concession Road 18-19, about 2 km west of North Woodslee. The Whelan Farm is rectangular, approximately 600 m by 650 m in plan area. The test plot was located in the southeastern corner of the property, in a long narrow area approximately 150 m by 10 m (Figure 7).

3.2 Climate, Vegetation and Land Use

Southwestern Ontario's climate consists of warm, humid summers and cold winters (Hunt, 1974). Annual average temperatures range from -7.8°C in January to 27.8°C in July. The mean temperature is approximately 9.2°C . The number of frost free days in the region is 165 ± 5 . Mean annual precipitation is about 825 mm (Ontario Ministry of Natural Resources, 1975).

The Eugene Whelan Experimental Farm is surrounded by farms that grow corn, winter wheat and soybeans. The Whelan Farm also grows these crops but they are cultivated in distinct plots separated by avenues of sod. All the wells installed for this project are located in unused plots.

The test site is located on the Essex Clay Plain which is a subregion of the more

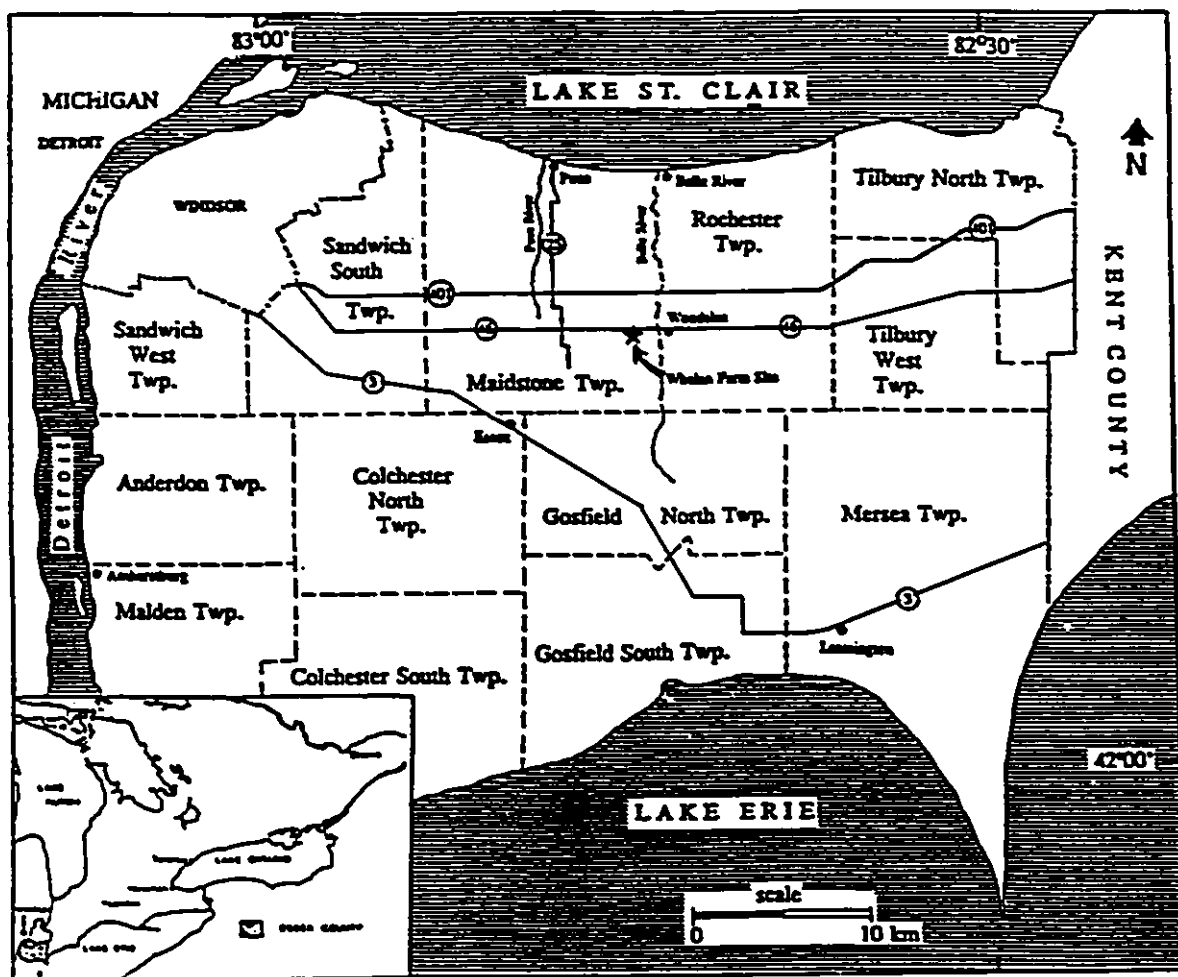


Figure 6. Location of study area.

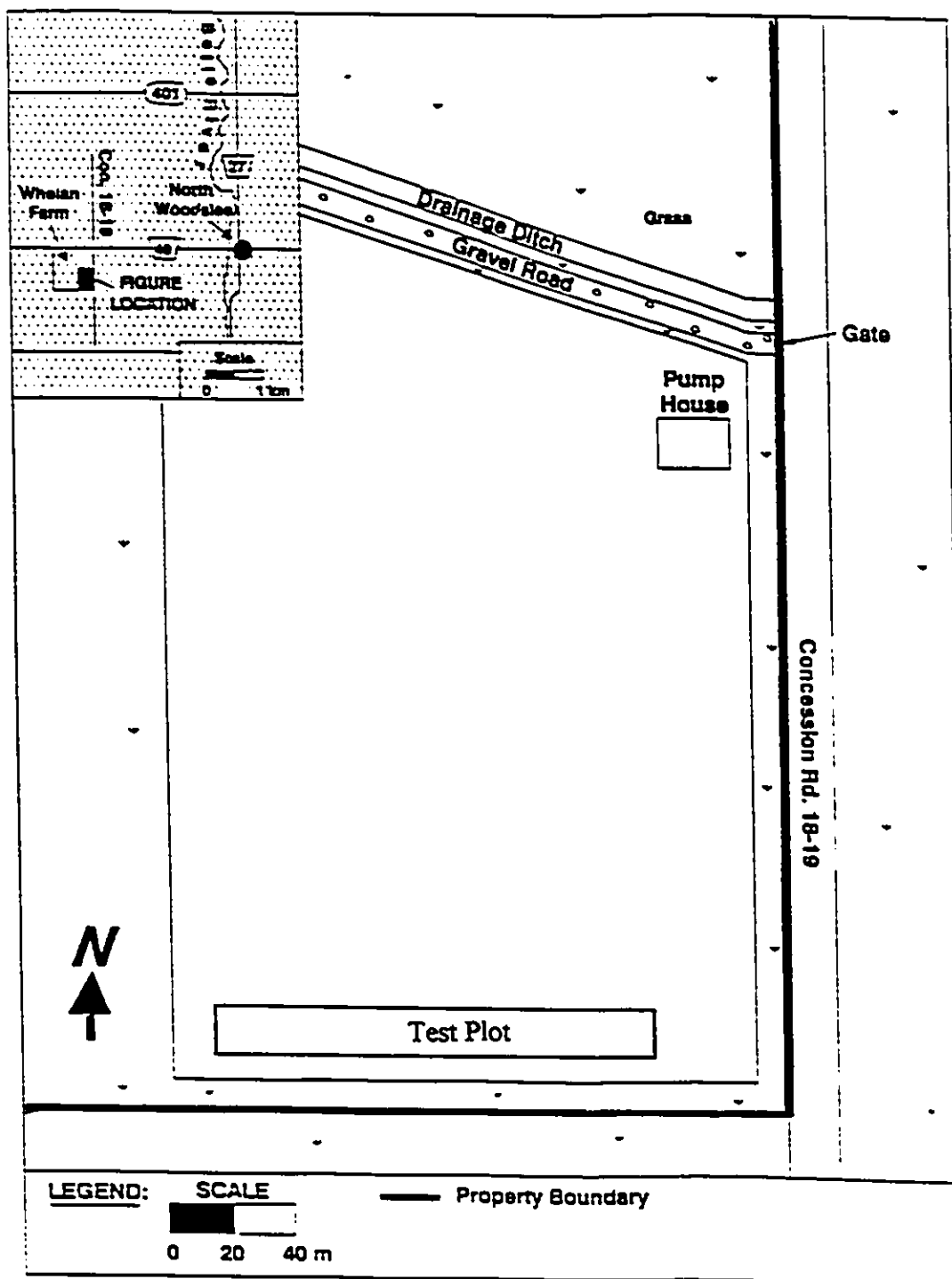


Figure 7. Southeast corner of the Whelan Farm Site showing the location of the test plot.

extensive St. Clair Clay Plain (Chapman and Putnam, 1984) (Figure 8). The Essex Clay Plain lies between Lake Erie and Lake St. Clair. It covers most of Essex County and the southwestern portion of Kent County. There is little surface relief and the slope and surface gradient are primarily northward toward Lake St. Clair under an extremely low gradient (approximately 0.1%). The clay soil has poor drainage and to provide proper conditions for crop growth, ditches and subsurface tile drains have been installed. Surveys denote the prevailing soil type as the Brookston clay loam (Chapman and Putman, 1984). The Brookston clay soil profile is described as having poor natural drainage and a fairly high organic matter content in the surface soil (Experimental Farms Service, 1946).

3.3 Geology

3.3.1 Bedrock Geology

Essex County is underlain by the eastern fringe of the Michigan Basin whose centre is located in central Michigan. The basin fill consists of sequences of Paleozoic carbonate, shale, sandstone and evaporite deposits. Grey shales and argillaceous limestones of the Middle Devonian Hamilton Group lie directly below the glacial overburden in the study area at a depth of around 30 m (Telford and Russell, 1981).

3.3.2 Quaternary Geology

The overburden in the St. Clair Clay Plain, including the study site, generally exceeds 30 m of Quaternary glacial drift. It has been described as a clayey silt till (Vagners, 1972), silty clay till (Desaulniers et al., 1981) and glaciolacustrine clay (Chapman and Putman, 1984).

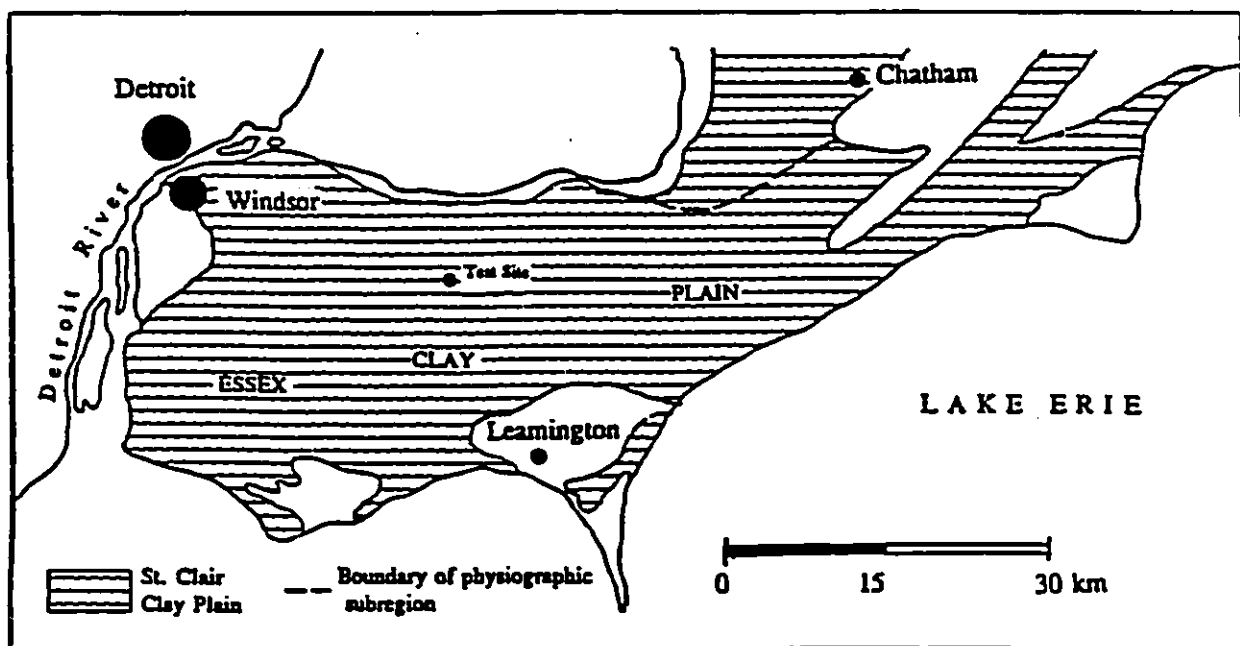


Figure 8. The Essex Clay Plain (Chiasson, 1992)

The exact origin of the till comprising the Essex Clay Plain is somewhat ambiguous. Lack of stratification tends to exclude a lacustrine origin and a normally consolidated state below a surface crust excludes a lodgement till origin (Vorauer et al., 1986). Chapman and Putman (1984) believe the clays were deposited by glacial Lake Wittlesey and subsequently by Lake Warren. Dreimanis (1961) attributes the extreme thickness of the till and its high clay content to more recent glacial reworking of older lacustrine deposits. Generally, the clay plain is referred to as a "water-laid till" (Dusseault and Vorauer, 1986) formed by glacial advance during the most recent (Wisconsin) event.

The tills throughout the St. Clair Clay Plain typically consist of 30-60% clay (generally 55-60%), 30-40% silt, 5-10% sand and less than 5% gravel (Harding, 1986). They contain carbonate and shale fragments from underlying bedrock as well as Precambrian rock fragments (Dreimanis, 1961).

The till profile is generally 2 to 4 metres of brown, weathered, desiccated and fractured till underlain by visually unweathered and unfractured blue/grey till. However, evidence exists to conclude that the upper few metres of the grey till in the St. Clair Clay Plain is fissured (Soderman and Kim, 1970; Brathwaite, 1988; and Ruland et al., 1991). The boundary between the brown and grey clay till is not necessarily a geologic boundary but is more likely a transitional boundary from oxidizing to reducing conditions (Ruland et al., 1991). The boundary is often not sharp and its depth varies throughout the region.

Discontinuous sand and gravel lenses have been intersected in boreholes at various depths throughout the clay plain. They occur in varying thicknesses of interbeds of silty sand with clay and minor amounts of gravel. This zone has been termed "the interbedded

zone" by consultants conducting subsurface investigations. M.M. Dillon Limited (1988) has attempted to correlate the interbedded zone throughout the St. Clair Clay Plain but its lack of continuity over a considerable distance leaves its origin debatable. It may be a remnant of dendritic stream channels or it may be the result of reworking of glaciodeltaic or floodplain deposits on to beach bars (M.M. Dillon Limited, 1988).

A fairly continuous layer of sand and gravel up to several metres thick underlies the grey till at the bedrock contact. This granular zone serves as the regions' freshwater aquifer.

3.4 Hydrogeology

Four hydrostratigraphic units can be recognized throughout the overburden in the St. Clair Clay Plain:

- 1) Brown weathered, fractured clay till
 - 2) Blue/grey unweathered till
 - 3) Sporadic interbedded zones of sand, gravel and clay
- and 4) Basal sand/bedrock aquifer.

In the immediate vicinity of the study area, water for domestic use is obtained largely from a municipal water distribution network constructed between 1982 and 1984 (M.M. Dillon Limited, 1988). On a more regional scale, however, groundwater is extensively used as evidenced by the abundance of wells listed in Ontario Ministry of the Environment (MOE) well records (1977).

Because of the very low topographic gradient and low permeability of the surficial clay soils, the main water-bearing formation throughout the St. Clair Clay Plain is the

basal sand/bedrock aquifer. Since it is likely that there is no physical barrier to flow between the basal sand and bedrock zones, they probably act as a single aquifer (M.M. Dillon Limited, 1988). This aquifer has been mapped as having a probable yield of 0.15 to 0.75 L/sec in the vicinity of the study area (Ontario Water Resources Commission, 1971). In Essex County, 69% of the MOE well records report water found in the bedrock and 25% report water found in the basal sand unit. Of these, the groundwater use is 43% domestic, 13% agricultural, 41% combined domestic and agricultural and 3% commercial and public (MOE Well Records, 1977).

M.M. Dillon Limited (1988) has determined the regional potentiometric surface for the basal sand/bedrock aquifer from static levels reported from MOE well logs. From the contours, it was determined that the gradient is approximately 0.001 northward, closely resembling the surface topographical gradient. Similarly, Essop (1986) concluded from groundwater elevations in dug wells that shallow groundwater also flows to the north under a gradient of 0.001-0.002 with local effects from rivers and streams.

Serving as a secondary source of groundwater in the area are shallow dug wells and cisterns. The water table is generally between 1 and 1.5 m in Essex County (Masoud Soultani, personal communication, 1993). MOE well records indicate that only one of 19 wells intersecting the shallow interbedded sand unit, which has been found at various depths throughout the St. Clair Clay Plain, derives water from that unit. This suggests that this unit is regionally not a significant aquifer.

M.M. Dillon Limited (1988) has summarized the hydraulic conductivity values for the brown till unit, the grey till unit, and the interbedded sand unit. An analysis of these

data reveals that the brown fractured unit is the zone of most active groundwater flow with hydraulic conductivities of at least one order of magnitude higher than the underlying grey unit throughout the St. Clair Clay Plain. Where present, the interbedded zone imparts a hydraulic conductivity value approximately two orders of magnitude higher than the brown fractured unit and provides a local conduit for groundwater movement (Chiasson, 1992).

Due to the very low hydraulic conductivities of the clayey soils and the lack of topographic relief of the St. Clair Clay Plain, groundwater velocities in the region are extremely slow to negligible. Groundwater in the deeper clay tills (blue/grey unit) is more than 8000 years old (Desaulniers et al., 1981) indicating that it is essentially immobile.

4.0 METHODS

This chapter describes the experimental procedures for both the laboratory and field studies. There were two objectives in the laboratory study. The first was to determine if freezing would reduce smearing during drilling. The second objective was to characterize the physical properties of the clay.

The field study involved the installation of piezometers using two techniques and subsequent single well tests. Sixteen piezometers were used as a control group. The control group was installed using the Overcored Shelby-Hole Method (OSHM) (D'Astous et al., 1989). Sixteen other piezometers were installed using the Frigid Insertion Method (Frigin), a modification of the OSHM, which employed liquid nitrogen to freeze the soil to prevent smearing.

4.1 Laboratory Tests

The laboratory study consisted of two parts: soil testing and constant head tests for K. The soil tests performed were Atterberg Limits, moisture content, hydrometer, and specific gravity. The tests were performed to obtain background data on the clay. Two sets of soil tests were conducted: one on the surficial fractured clays to determine soil properties at the surface and the second on clays obtained from about 15 m below the surface. The deeper clays were obtained from Khaled Chekiri (a Ph.D. candidate in Civil and Environmental Engineering, University of Windsor) out of one of the split spoon samples from his drilling program approximately 7 km East of the Whelan Experimental Farm.

4.1.1 Constant Head Test

The objective of the constant head test was to develop a reproducible laboratory experiment that would determine whether pre-freezing of a fractured clay soil column would reduce smearing of fractures during drilling. In this test, a test cylinder was first packed with clay until it was half full. A filter paper was laid across the column to simulate a fracture, and then more clay was added.

Two sets of tests were run on the packed cylinder. In the first set of tests, a hole was drilled into the clay in the cylinder while the cylinder was at room temperature. A constant head test was then conducted on the cylinder to determine the flow through the fracture. In the second series of tests, the cylinder was frozen prior to drilling the hole and constant head tests were again run to determine the flow through the fracture.

The apparatus used in the constant head tests was constructed in the Central Machine Shop, University of Windsor. It consisted of six parts: two threaded caps with threaded holes to attach tubing connectors, two short cylinders made of aluminum (threaded, to fit into caps), and two halves of a longer cylinder made of plexiglass which fit in between the short cylinders (Figures 9 and 10). Figure 11 is an exploded view of the cylinder that shows all the pieces. The dimensions of the fully assembled cylinder are 10 cm in diameter (inside) and 18 cm in height. The apparatus was constructed in pieces because it was originally designed for actual collection of the sample. During the investigation, however, remolded columns were used to minimize water bypass during the K test.

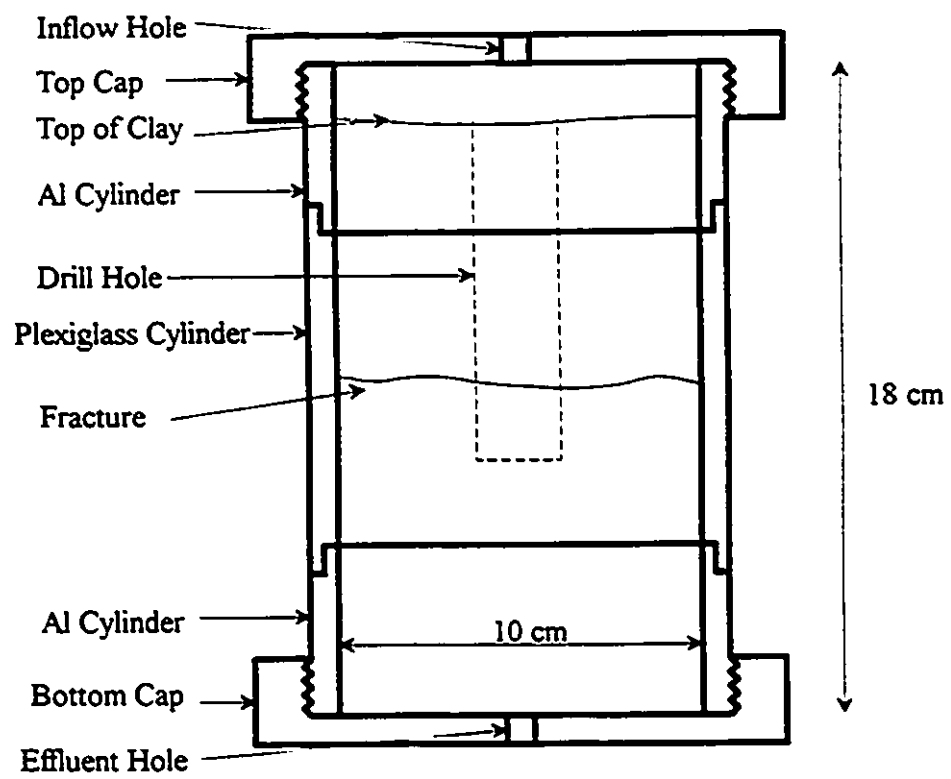


Figure 9. Schematic diagram of the clay column cylinder.



Figure 10. Photograph of the clay column cylinder.

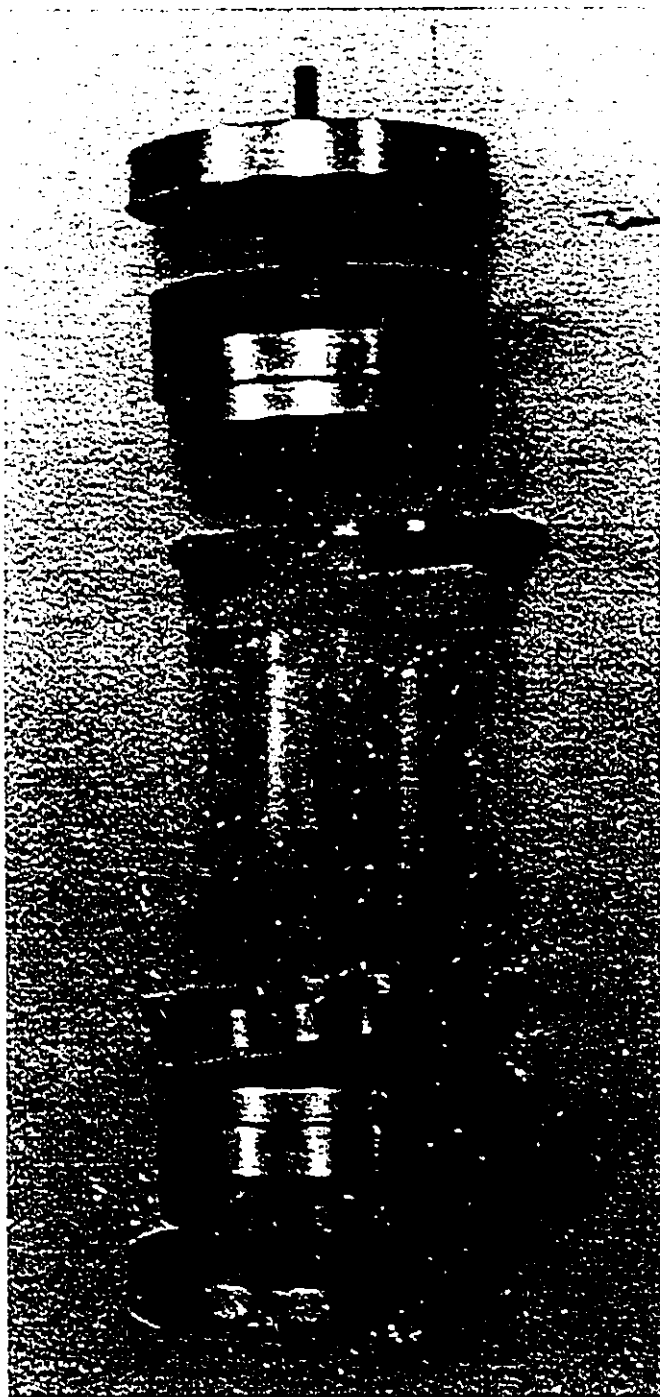


Figure 11. Photograph of the clay column cylinder (disassembled).

The cylinder was assembled as follows. One of the threaded caps was used as the base. Its threads were greased with Phil Wood's grease, a high quality bicycle grease, and one of the short threaded cylinders was screwed on. A clear silicone caulk was smeared on the contact points of the plexiglass cylinder halves, which were then pressed together. More silicone was used to seal the connection between the plexiglass cylinder and the short aluminum cylinder at the base. The second short aluminum cylinder was attached to the plexiglass in the same manner. Two pipe compression straps were then placed around the top and bottom of the plexiglass portion and tightened to secure the section as the silicone caulk dried. The compression straps were left in place to keep the plexiglass from coming loose. After the cylinder was packed with clay (described in the next section) the threads of the top cap were greased and the cap was screwed on to the top of the cylinder. A nylon tubing connector was screwed into the threaded hole in the top cap and sealed with teflon tape.

4.1.2 Preparation of Simulated Fractured Clay Soil

The clay used in the cylinder was obtained from Shelby tube samples collected by Khaled Chekiri, a doctoral candidate in Environmental Engineering at the University of Windsor, during the course of his field work (August, 1992). This clay was used instead of the surficial clay at the study site because it was unweathered, plastic, and all the physical properties of the clay were known. The surficial clay at the field site was not used because it was weathered, dry, and difficult to form into the cylinder. In addition, the test was performed to see if the pre-freezing worked, so the clay used in the cylinder was of secondary importance.

After the cylinder was assembled (except for the top cap), it was ready to pack with clay. A fine metal screen was first placed over the threaded hole in the bottom cap. This prevented the clay from pushing through the hole. A piece of Number 3 filter paper was cut to fit the diameter of the cylinder and placed over the screen. Two more pieces of this filter paper were cut into strips about 2cm wide and placed vertically along the edge of the cylinder. These two strips served as conduits of water from the fracture to the discharge hole. These strips were placed opposite each other and in contact with the filter paper forming the fracture and the filter paper on the bottom of the cylinder.

Golf-ball-sized pieces of clay were flattened, placed inside the cylinder, and pressed in place by hand. The moisture content of the clay, at 20.5%, was halfway between the liquid limit and the plastic limit (all the soil properties are listed later). This moisture content was ideal for packing the cylinder. The clay was neither too stiff to form easily nor too watery. Care was taken not to dislodge the filter paper and to prevent air pockets and short circuits along the walls of the cylinder. Clay was added this way until the cylinder was half full of clay. At this point the surface of the clay in the cylinder was flattened and the artificial fracture was constructed.

The artificial fracture consisted of two pieces of filter paper. Sand was considered for the fracture but it would have penetrated into the clay. This would have made the thickness of sand needed to keep the fracture open difficult to determine. Maintaining a constant fracture aperture would also have proven difficult. Two pieces of filter paper were used because the clay would clog the paper to a certain extent. Having two pieces of filter paper ensured an open conduit for the water.

The two pieces of filter paper, the first Number 3 and the second Number 4, were cut to fit the diameter of the cylinder. The Number 3 paper was placed in first and the two strips of filter paper (placed vertically before the clay was added) were folded over on to the Number 3 paper. The disk of Number 4 paper was then placed over the first circle and the folded over strips of filter paper. At this point, more golf-ball-sized pieces of clay were added (as above) until the clay was about 3cm from the top of the cylinder. The packed cylinder was now ready for either freezing or drilling.

A proctor hammer was originally going to be used to pack the clay, using 25 blows per lift, which is standard Proctor density procedure. It was found that the clay was liquid enough to flow with each blow of the hammer. This moved the filter paper around a great deal and the contact between the pieces of filter paper could not be ensured. At the point where the fracture was installed, the hammer blows tore the filter paper keeping the fracture open. This made it impossible to keep a fracture open and packing the cylinder by hand became the only option.

The density of the packed clay was not a concern. It was more important to ensure that no short circuiting of water occurred. The moisture content of the clay was high enough so that careful packing of the cylinder by hand prevented short circuits.

Chekiri (personal communication) reported that the clay soil used in the constant head test had a specific gravity of 2.77, a moisture content of 20.5%, a 17.7% plastic limit, and a 23.9% liquid limit. The grain size distribution was 42% clay, 33% silt, 21% sand and 4% gravel. The grain size distribution values were obtained from a combination of a hydrometer and sieve analyses. The limits of the grain size divisions were those of

the U. S. Department of Agriculture (Das, 1985).

4.1.3 Drilling Across Fractures

To simulate auger drilling in the laboratory, the prepared cylinder was drilled using a 7/8" drill bit in a drill press. The rotational speed of the drill was 100 rpm. This speed was used because it was the closest (slowest) speed that the drill press could attain to the rotational speed of the augers on a drill rig. The clay packed cylinder was centred under the drill, and the rotating drill bit was pressed slowly through the clay to 2 to 3 cm through the artificial fracture. The drill bit was slowly raised out of the cylinder while it was still rotating. The hole was then cleared of any clay pieces that may have fallen into it.

4.1.4 Constant Head Test Procedures

Figures 12 and 13 show the constant head test apparatus. A 30 L ralgene reservoir was used as a source of water in the constant head test. The diameter of the reservoir was large enough and the amount of water flowing through the clay small enough so that the level of water in the container essentially did not change. The water used in the constant head tests was deionized water with a small quantity of dissolved silver nitrate to prevent biological activity (Dr. P. Hudec, personal communication).

The reservoir had a valve assembly at the bottom for regulating liquid flow. The reservoir was placed on an inverted five gallon pail to increase the head difference between the surface of the water and the outlet at the base of the cylinder. A rubber hose was attached to the valve assembly at the base of the reservoir and the hose's other end was attached to the tubing connector at the top of the cylinder. The cylinder was placed inside a five gallon pail to collect the water that ran through it. The cylinder rested on

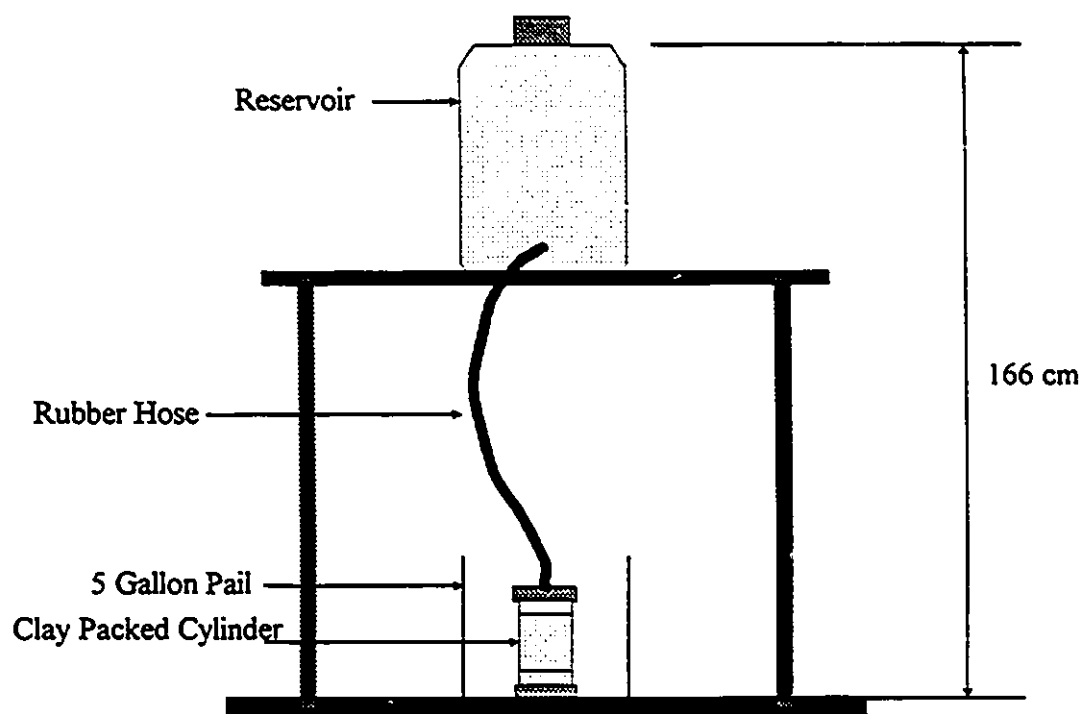


Figure 12. Constant head test apparatus.

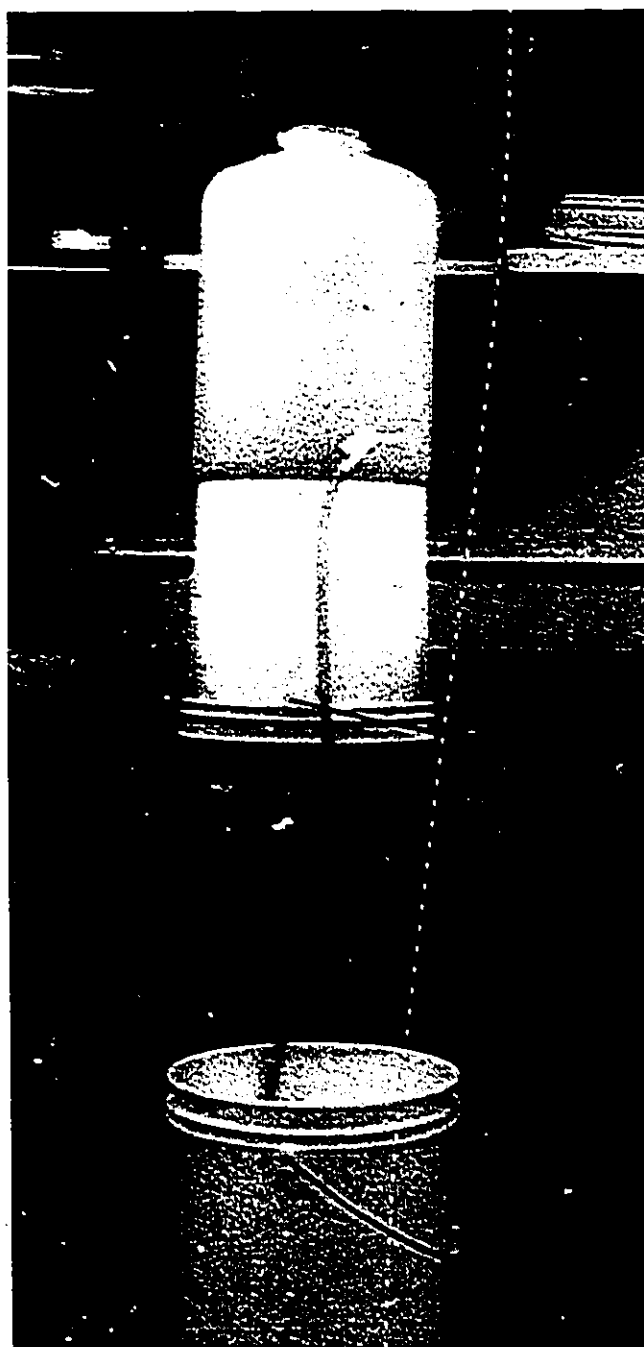


Figure 13. Photograph of the constant head test apparatus.

three coins to raise it above the bottom of the pail to ensure liquid could escape through the hole in the bottom cap. The total height of the water column was 166 cm.

At the start of the constant head test, the valve on the water container was opened and water was allowed to flow into the top of the cylinder. Any air trapped in the top of the cylinder was purged by rapidly squeezing the hose and letting it go. This caused pressure/water surges that forced water into the top of the cylinder, displacing any trapped air. Once all the air was displaced, the time was noted and the apparatus was left for 24 h.

After 24 h, the valve on the water container was closed, and the cylinder was taken out of the pail on the floor and placed on the counter. The water in the pail was poured into a graduated cylinder and its volume measured. The pail was placed back on the floor, the coins repositioned, and the cylinder replaced in the pail. The water in the graduated cylinder was poured back in the top of the container of water. If the water in the container was below a set level, more deionized water was added. The valve was then opened and the time noted, and the cycle started again. This cycle continued from four to six days. Since the time the valve was opened and the volume of water collected was known, the discharge rate for the time period could be calculated.

4.1.5 Field Soil Properties

The weathered clay at the field site had a specific gravity of 2.73. The moisture content was 17.5%. The Atterberg limits were 23.9% for the plastic limit and 38.1% for the liquid limit. Because the native moisture content was below the moisture content for the plastic limit, the natural condition of the clay would be classed as a semi-solid (Das, 1985).

The results of the grain size distribution were 33% clay, 39% silt and 28% sand and gravel. The limits of the grain size divisions were those of the U. S. Department of Agriculture (Das, 1985). The values for sand and gravel were combined because there was no sieve analysis done on this sample.

The weathering of the clay made it difficult to work with. The sample had to be broken up by hand because the clay was very hard. Breaking up the clay mechanically might have crushed larger particles, skewing the results. Water was added to the clay to facilitate the reduction of the larger clumps of clay. All the gravel was taken out before the hydrometer was run (the mass of the gravel was included in the calculations) and before the Atterberg limits were determined.

4.2 Field Study

The field study was conducted to determine the effects of freezing clay prior to drilling. It was hoped that freezing the clay would reduce smearing caused by the installation of the piezometer. If that was the case, more accurate K values would be obtained.

The field study consisted of single well (slug) tests conducted on thirty two piezometers. The piezometers were installed two different ways. The Frigin method was used to install the first set of sixteen piezometers, on February 1, 1993. The second set of sixteen piezometers were installed using the OSHM method on March 3, 1993. The spatial relationship between the piezometers is shown in Figure 14.

Recovery data were recorded until the middle of May, 1993. One slug test was conducted in each of the months of June, July, and August, 1993.

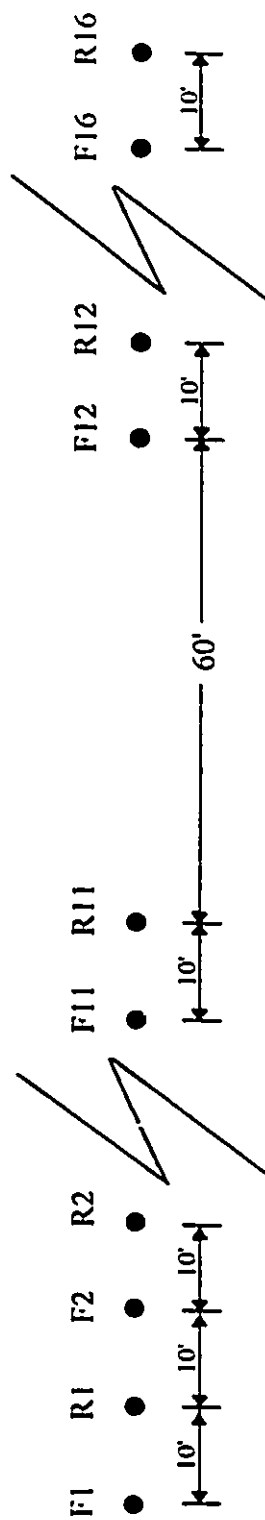


Figure 14. Spatial orientation of test piezometers.

4.2.1 Preparation of Shelby Tubes

The drilling technique employed two sizes of Shelby tubes to make the hole for the screened portion of the piezometer (D'Astous et al., 1989). One of the sizes was the standard three inch Shelby tube. The other size would normally be a standard two inch Shelby tube, but due to budgetary limitations, Shelby tubes were constructed at the University of Windsor. These custom-made tubes were fabricated from 1 7/8 inch diameter automobile exhaust pipe (Figures 15 and 16).

The "standard" three-inch Shelby tubes were modified to accommodate the OSHM method of piezometer installation described later in the chapter. The modification entailed the removal of the standard outside bevelling and the machining of an inside bevel. The outside bevelling was removed with a pipe cutter. The cut end was then filed to produce the inside bevel (Figures 4 and 5). The length of the three-inch Shelby tubes varied by about 10 cm because they were used in previous research and in some cases were bent or otherwise damaged.

The small diameter Shelby tubes were constructed from 50 cm long sections of exhaust pipe. Both ends were filed to remove any burrs. One end was additionally filed to create an outside bevel. The other end had a 1cm hole drilled through it to accommodate a bolt for connecting the constructed Shelby tube to a piece of a standard two-inch Shelby tube (this was done because there are standard connector pieces that attach the Shelby tubes to the drill rigs). The constructed Shelby tubes bolted inside the two inch Shelby tube which was then bolted on to the drill rig.

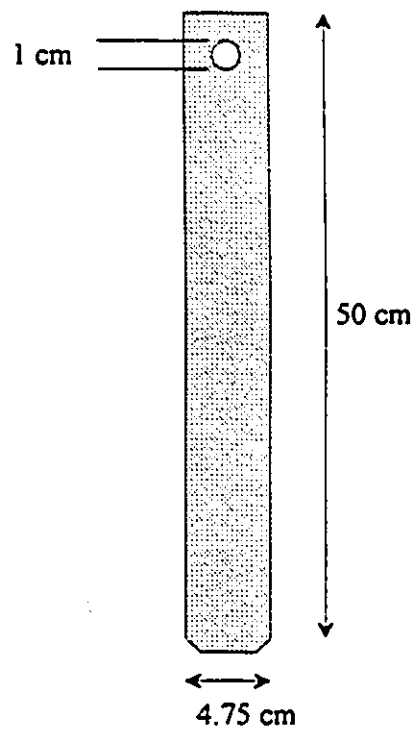


Figure 15. Small Shelby tube schematic.



Figure 16. Photograph of the small Shelby tube.

4.2.2 Overcored Shelby-Hole Method

The overcored Shelby-Hole Method (OSHM) was developed to reduce the smearing of fractures around the screened section of piezometers installed in fractured clay tills (D'Astous et al., 1989). It is an adaption of a concept of augured-hole reaming used by Lafleur and Giroux (1983). Reaming refers to the removal, using a wedged cutting tool, of the smear and shear on the borehole wall. In this case, it was done using two Shelby tubes of different diameters in a manner originally proposed by M. B. Dusseault (personal communication to D'Astous in 1986, from D'Astous et al. 1989).

Initially, a borehole is advanced to just above the desired screen height using a hollow stem auger. At this point, the plug is removed from the bottom of the auger and the first Shelby tube is hydraulically advanced into undisturbed soil, creating a pilot hole. The pilot hole is important because the second Shelby tube is inside bevelled. The inside bevel of the smaller Shelby tube forces the clay into the centre of the tube. It is important that the clay is moved toward the centre of the tube because moving it outside would disturb the surrounding clay, possibly closing fractures. The second Shelby tube is pulled out without rotation to further avoid smearing.

All of the boreholes were drilled using this technique. The method was the same for the boreholes where the clay was pre-frozen. Dominion Soil Investigation Inc. (Windsor) drilled the holes for all of the piezometers. The drill rig used was a CME 750, four wheel drive power auger machine manufactured by Central Mining Equipment Limited, and equipped with two sizes of hollow stem augers, 82 and 108 mm inside diameter. The 82 mm hollow-stem augers (six inch outside diameter) were used

exclusively to drill the boreholes.

After the borehole was completed, the piezometer was installed. The screen and casing were prepared beforehand. They consisted of a five-foot section of 1 1/4" diameter, schedule 40 casing, and a two-foot section of 1 1/4" diameter, schedule 40, number 10 screen (Figure 17). The casing and screen were purchased from Environmental Systems Canada Limited at cost through Dominion Soil Investigation Inc. The casing was purchased in ten-foot lengths and the screen was purchased in five foot lengths. Cutting the casing and screen required that half of the casings and screens be glued together. A PVC cement was used for this task. The other half of the screens and casings were screwed together as they were designed to. A slip on cap was glued to the open end of the screen to prevent any material from entering the piezometer.

The screen and casing were placed in the hole after it was drilled. Most of the piezometers were screened between 157cm and 205cm below ground level (BGL). The piezometer was completed by filling the annulus with a coarse sand to 5 cm above the screen. A 15 to 20 cm plug of bentonite chips was then poured into the annulus, and the rest of the hole was filled with cuttings (Figure 18). The heights of sand and bentonite were measured with pre-marked rods. The rods were also used to tamp down the sand, bentonite, and cuttings. Any casing that stuck up above the ground more than about 12 cm was cut off with a hacksaw, and the cut off length recorded. All the piezometers were capped with a PVC slip cap.

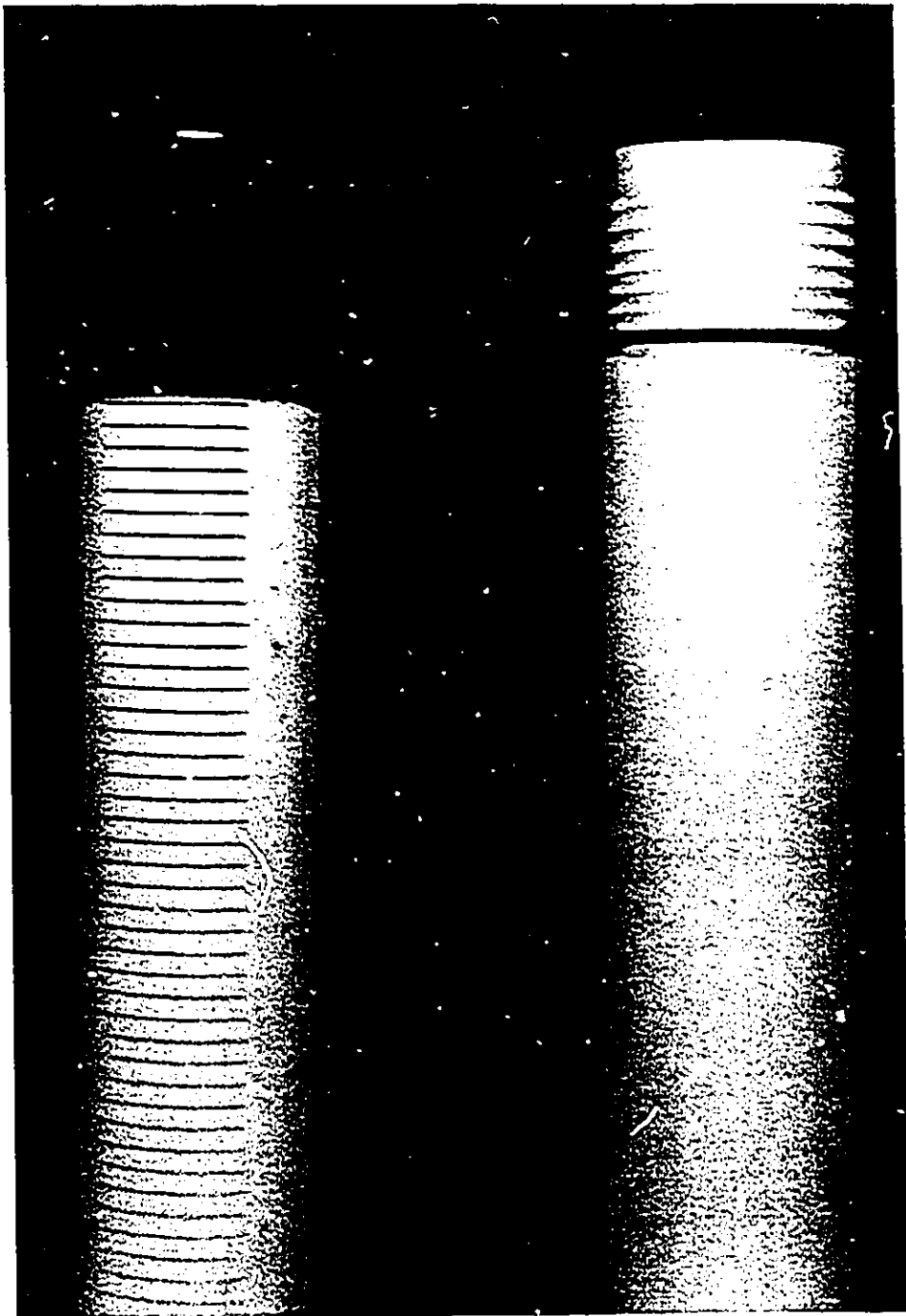


Figure 17. Photograph of piezometer screen and casing.

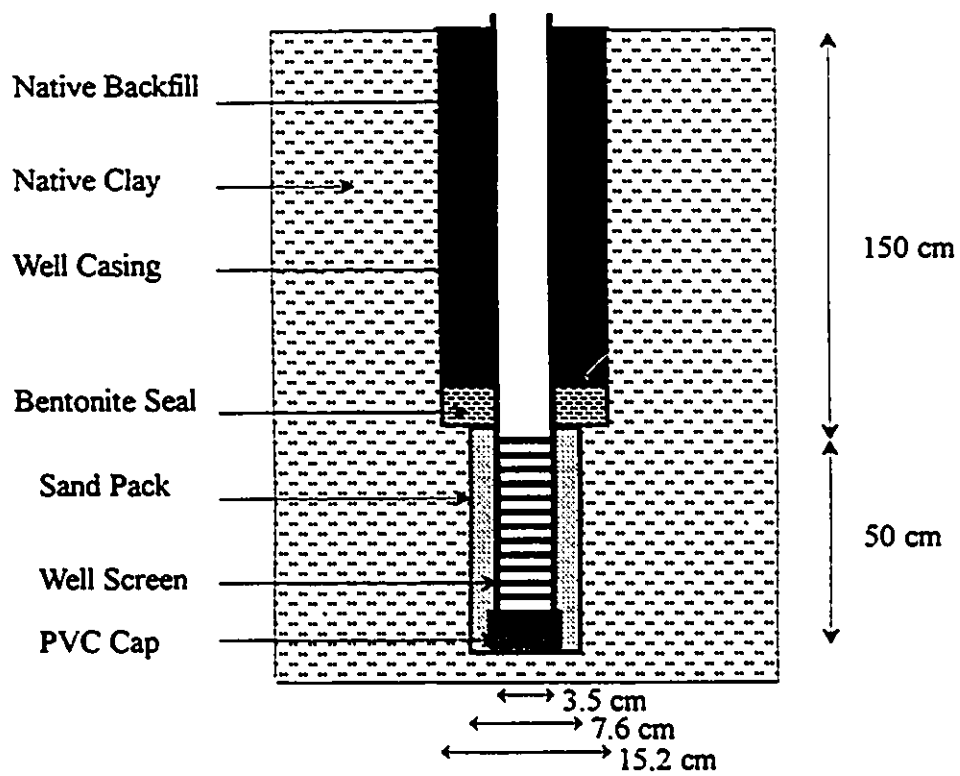


Figure 18. Typical piezometer installation.

4.2.3 Frigid Insertion Method

The modification made to the OSHM method is freezing the clay prior to completion of drilling. This was accomplished by adding liquid nitrogen after the first Shelby tube was removed but before the second tube was pushed in. This allowed the nitrogen to freeze the area directly affected by the insertion of the second Shelby tube. The rest of the well installation was the same. This method is referred to as the Frigid Insertion Method (Frigin). The Frigin piezometers were spray painted to distinguish them from the OSHM piezometers.

4.2.4 Single Well Test Procedures

After all the piezometers were installed, they were allowed to stabilize to static water levels. Recovery took three to four months since the piezometers were installed in the winter and the spring thaw contributed to water level rise. The static water level needs to be determined in order to calculate the head difference when a slug of water raises the level of water in the piezometer casing, as described in Section 2.7.

The single-well tests conducted are generally called slug and bail tests. Slug tests are so termed because a known volume of water is added to the piezometer to raise the water level above static. Bail tests remove water to lower the water level in the piezometer below static. The recovery data from either of these procedures is used to calculate the hydraulic conductivity. In this investigation, slug tests were used exclusively because there was not enough water available in the wells to perform bail tests (i.e. the static level was below the top of the screen).

At the beginning of each set of slug tests, the water level was measured relative to

ground level in each piezometer. Then water was added quickly to each piezometer to bring the water level up to the ground surface. The water level decline was recorded at regular intervals relative to the top of the piezometer casing. The water levels were corrected to ground level for comparison. Water level measurements were taken for four weeks. The water levels were taken more frequently in the first and second week than in the third and fourth weeks. Three or four measurements were taken in the first week after the test began, two measurements in the second week, and one at the end of the next two weeks. The decreasing frequency of observations reflects the decreasing rate of change of the water levels in the piezometers. The decreasing rate of change is the result of the water level in the piezometers returning to static. This decreases the hydraulic head in the piezometer, so the force driving the water out of the piezometer progressively diminishes.

The hydraulic conductivity value at each piezometer was determined using Hvorslev's method, described in detail in Section 2.7.

5.0 RESULTS AND DISCUSSION

5.1 Constant Head Test

The constant head test was conducted in order to determine if freezing clay prior to drilling would reduce smearing. This was accomplished by making an artificial fracture in a cylinder packed with clay, freezing the whole cylinder, and then drilling into the frozen clay through the fracture. After drilling through the frozen clay, the cylinder was left to thaw, and a constant head test was conducted to determine the hydraulic conductivity. Constant head tests were also conducted on cylinders that were prepared the same way but not frozen. The hydraulic conductivities of both types of tests were then compared to see if there was a difference.

5.1.1 Constant Head Test Results

The results of the constant head test suggest that freezing the clay prior to drilling reduces the smearing of clay. Three tests were conducted for each drilling method (frozen and room temperature). Table 1 shows the data and results of the tests. There is a significant difference in the discharge rates between the two types of tests conducted. The discharge rates of the tests where the soil was frozen before drilling were much greater than those in which the soil was not frozen. The maximum discharge rates for the tests where the clay was frozen prior to drilling and where it was drilled at room temperature are 387.7 and 15.4 mL/hour, respectively. The minimum discharge rates were 15.2 and 5.7 mL/hour, respectively. The geometric mean of the hydraulic conductivities for the clay packed cylinders that were not frozen before drilling was 3.8×10^{-5} m/sec. The geometric mean of the hydraulic conductivities for cylinders that

Table 1. Constant head test results.

	Time Interval (hours)	Water Volume mL	mL/hour	K (m/sec)	Ave. K (m/sec)	Geo. Mean (m/sec)
<u>Room Temperature</u>						
Test 1	24	334	13.9	5.3E-05	4.9E-05	3.8E-05
	24	370	15.4	5.9E-05		
	18.67	246	13.2	5.1E-05		
	24	216	9.0	3.5E-05		
Test 2	24	307	12.8	4.9E-05	3.4E-05	
	24	232	9.7	3.7E-05		
	23.5	178	7.6	2.9E-05		
	23.5	133	5.7	2.2E-05		
Test 3	24	234	9.8	3.7E-05	3.2E-05	
	24.5	231	9.4	3.6E-05		
	23.5	218	9.3	3.6E-05		
	24	205	8.5	3.3E-05		
	24	188	7.8	3.0E-05		
	22.5	159	7.1	2.7E-05		
	24	150	6.3	2.4E-05		
<u>Frozen</u>						
Test 1	23	2190	95.2	3.7E-04	1.7E-04	2.6E-04
	24	975	40.6	1.6E-04		
	24	593	24.7	9.5E-05		
	25	380	15.2	5.8E-05		
Test 2	24	1672	69.7	2.7E-04	1.5E-04	
	24	923	38.5	1.5E-04		
	25	642	25.7	9.8E-05		
	22	487	22.1	8.5E-05		
Test 3	7.75	3005	387.7	1.5E-03	7.4E-04	
	16	4340	271.3	1.0E-03		
	23.6	5125	217.2	8.3E-04		
	24.5	4180	170.6	6.5E-04		
	24.8	3570	144.0	5.5E-04		
	23.75	2910	122.5	4.7E-04		

$$K = (Q \cdot L) / (A \cdot h \cdot t)$$

Q = volume of water collected

L = length of cylinder

A = cross sectional area of cylinder

h = head difference

t = duration of collection of water

were frozen before drilling was 2.6×10^{-4} m/sec.

5.1.2 Statistical Analysis For The Constant Head Test

No statistical comparisons were performed on the results of the constant head tests. The main reason for this is the number of tests conducted. The probability that an estimate differs by less than an arbitrary small error approaches unity as the sample size increases to infinity (Kennedy and Neville, 1986; Wallis and Roberts, 1963). In other words, the smaller the sample, the less accurate are the inferences drawn from it. Because there were only three tests performed of each type, it was decided that a direct comparison of the results was adequate to ascertain a difference.

The average hydraulic conductivity was calculated for each of the tests. The total flow through the cylinder and the total time that the test was run were used to calculate the average K. Comparing these values between the two types of test show a difference of approximately one order of magnitude. Since the only difference between the two types of tests was freezing the clay prior to drilling, the difference in results appears to be due to reduced smearing at the fracture.

5.1.3 Constant Head Test Discussion

The discharge rate and the resulting hydraulic conductivity values for the third test where the clay was frozen prior to drilling were significantly higher than the other two tests where the clay was frozen. The reason for this was not apparent. The differences between this test and the other two were: the cylinder was completely disassembled and new silicone applied, some of the clay was replaced with new clay, the clay was thoroughly reworked and the water in the reservoir was changed.

The cylinder was disassembled because the seals failed. Some of the clay was replaced because it started to scale after the second test and because some black staining (described later) on the clay was removed. The clay was reworked to mix the old and new clay. The water in the reservoir was changed because it was lost when the seals failed on the cylinder. When the water was replaced the precipitate was removed which would have reduced the rate at which the fracture clogged up.

All the tests show a gradual decrease in discharge rate with time (Figures 19 and 20). This decrease is probably due to siltation. Because of the high rate of flow through the fracture, any sediment suspended in the water would be filtered out by the filter paper in the fracture opening. Over time, as more material was filtered, the ability of the filter paper to pass water would be reduced. The clogging of the filter paper would account for the reduced flow over time. A precipitate formed in the water reservoir and this precipitate in suspension may have contributed to clogging the artificial fracture. Biological activity should not have been a problem because the water was treated with silver nitrate.

Black staining was noticed on the filter paper, on the clay in contact with the water, and on the aluminum of the test cylinder. The aluminum was roughened beneath the black stain. These conditions were thought to be reactions with the silver nitrate. The water also reacted with the coins supporting the cylinder, leaving a black precipitate. Any water that was lost was replaced with tap water, which may have eventually reduced the effectiveness of the silver nitrate.

The test was not actually a true constant head test. A large (wide) container was

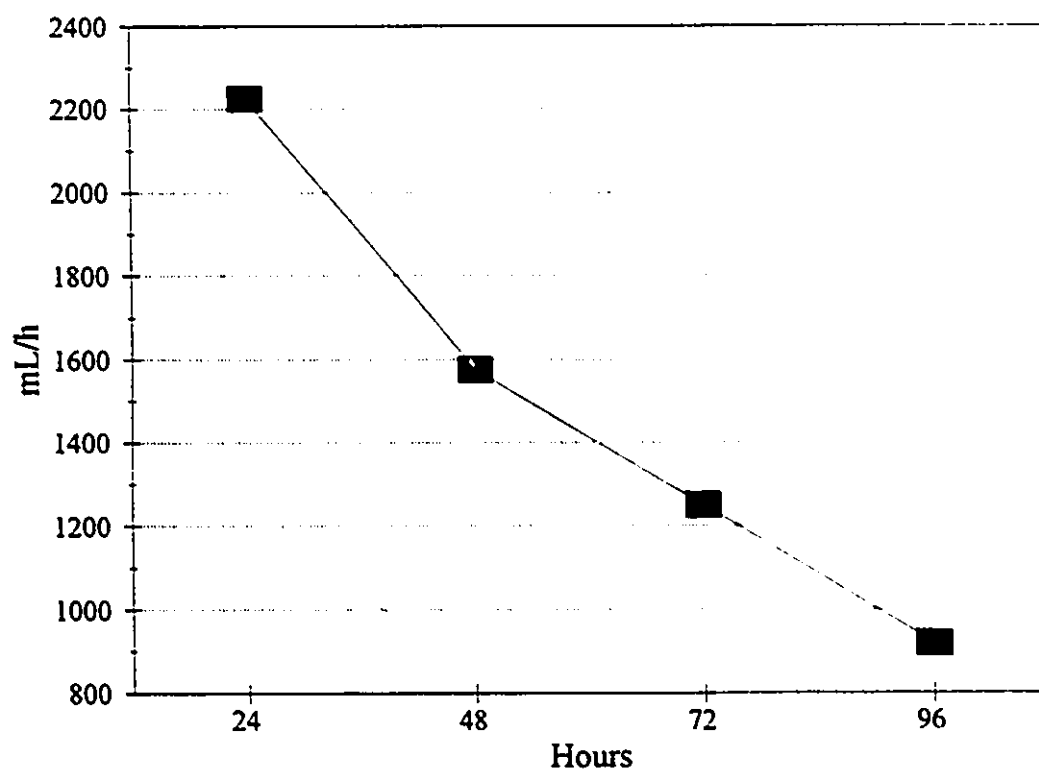


Figure 19. Average discharge rate versus time for the tests where the cylinder was frozen prior to drilling.

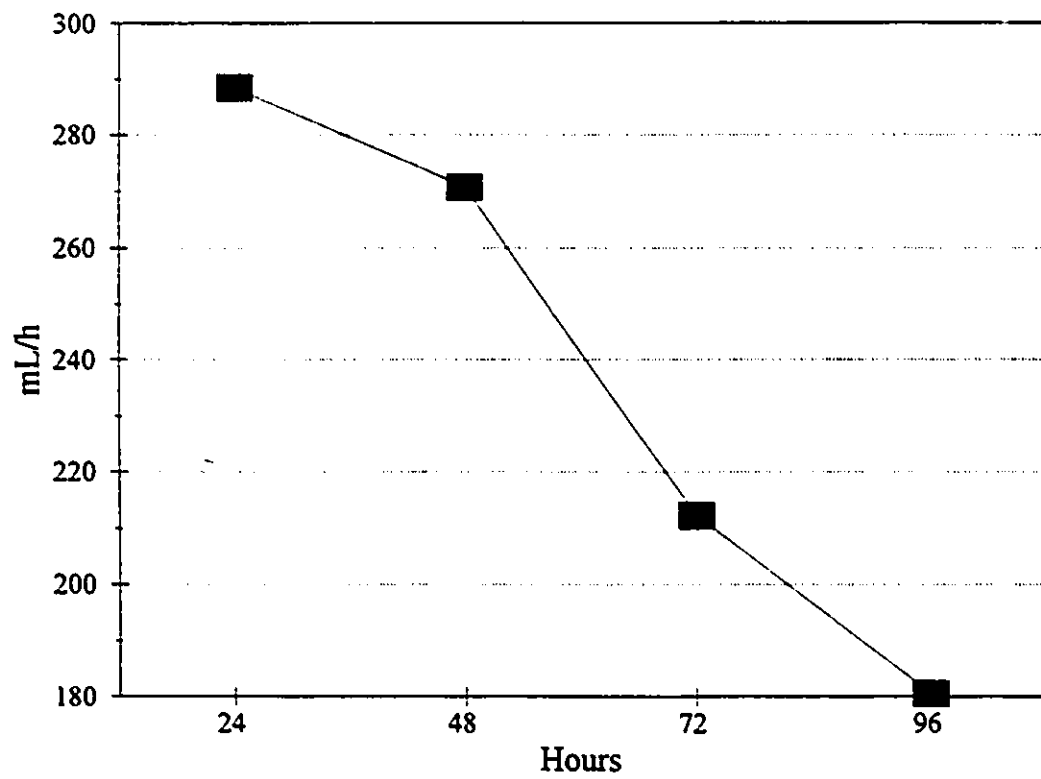


Figure 20. Average discharge rate versus time for the tests where the cylinder was at room temperature when drilled.

used as the water supply but there was no provision for maintaining a constant head . The large surface area of the reservoir, however, minimized changes in the head. During the tests where the hole in the clay was drilled at room temperature, the change in head was at most 0.5 cm in 24 hours. This caused a head difference of 0.3%. The first two tests in which the clay was frozen before drilling, resulted in head changes of less than 3 cm, a difference in head of 1.8%. The last of these tests resulted in a maximum head change of 15 cm. The difference in head was 9%.

5.2 Field Observations

The clay soil over most of the test site is homogeneous with respect to soil properties, location and frequency of fractures, and depth of the weathered zone. One sand lens was intersected during drilling. At the eastern end of the line of wells, there is an area of silty sand 1 m below the surface extending to an unknown depth. It is a light tan colour, as compared to the brown weathered clay. The corn in the adjacent plot and the surrounding ground cover that overlies this soil grew faster, taller, and did not wilt as much during dry spells as the foliage in other areas. This effect has been noted by the employees of Agriculture Canada at this site for many years now (Masoud Soultani, personal communication).

A similar soil was encountered at a different location on this site by Chiasson (1992). He identified it as the interbedded zone, which is discontinuous through the St. Clair Clay Plain. The grain size analysis he performed indicates that it consists of 76% sand and gravel, 12% silt and 12% clay. He encountered the interbedded zone about 2.3 m below the ground surface.

All the soil in the smaller Shelby tubes was extracted, examined and photographed. The main characteristic sought was if the borehole intersected any fractures in the screened portion. Nearly all the wells showed well-defined oxidation lines indicative of fractures (Figure 21). These oxidation lines are easily distinguishable by reddish-purple, yellowish-brown and greyish-white coatings associated with chemical alteration haloes. Reddish coatings are likely iron hydroxides (Quigley and Ogunbadejo, 1973) and whitish coatings are apparently calcite (Ruland et al., 1991). Some fractures were large enough that the sample broke along the fracture plane.

Generally, there was only one main fracture in the soil core that had an oxidation halo. The fractures around this main fracture were also oxidized. The fractures were concentrated around the main fracture. Other fractures in the core were smaller and difficult to see. Many did not have oxidation haloes and the process of removing the clay from the Shelby tube made it difficult to determine if the fractures existed before the clay was extracted. There was no preferred orientation of the fractures.

5.3 Static Water Table Conditions

The water table depth at the test site varied with the season and the weather. Variables such as rainfall, evapotranspiration, and location of surface water all affect the water level. Monitoring of the water table began as soon as the first set of wells was installed in early February, 1993. These wells were the sixteen wells installed using the Frigin method.

The installation of the OSHM wells occurred in early March. The water table rose slowly in all the wells until the middle of March. At this time, the ground had thawed

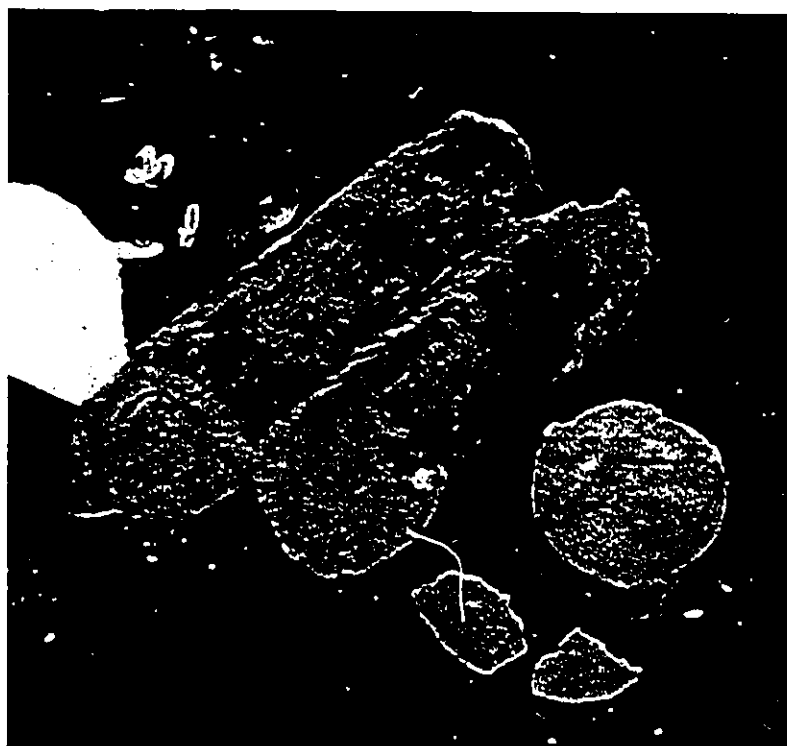
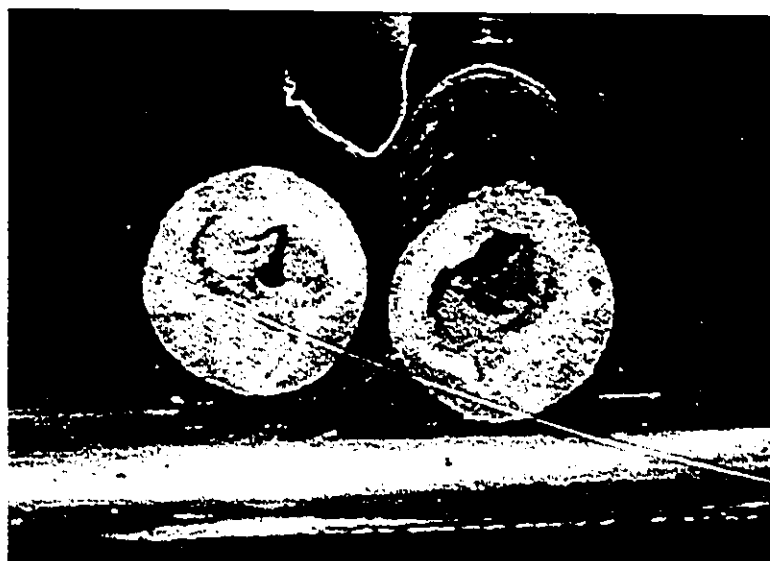


Figure 21. Photograph of fractures in clay removed from Shelby tube.

through to the unfrozen ground underneath and the water levels rose more quickly. The water level rose to about 40 cm below the ground surface in most of the Frigin piezometers by late April, the highest the water table recorded during the study (Figure 22). The water table slowly declined throughout the summer, falling to more than 200 cm below the ground surface (the deepest wells were 205 cm deep) by August.

The static water level dropped slowly during the summer in response to low rainfall and evapotranspiration by the crops. Since the water table dropped throughout the summer, it was not possible to use a single level when conducting the Hvorslev tests. The static water level used in the calculations was taken from the piezometer that had the lowest water level at the end of the test (140 cm below the ground surface in the June test). For the second and third tests, a static level of 202 cm was used. This was the maximum depth of the wells (some of the wells went dry in the second and third tests).

F1, the first piezometer in the Frigin series, intersected a large fracture network. At the start of the second slug test, the piezometer accepted more than 30 L of water (no exact count was kept) and the water level in the piezometer could not be brought up to the ground surface for any length of time. Since none of the piezometers had a volume of more than 3 L, the water had to be draining into a large fracture network or sand lens. The shape of the recovery plot of this piezometer is similar to plots of other piezometers and the K value obtained from this well did not prove to be an outlier (outliers are discussed later). This piezometer was used to determine static level until it went dry (at 147 cm). Other piezometers drained quickly when water was added at the start of the second and third slug tests, but not nearly as quickly as F1.

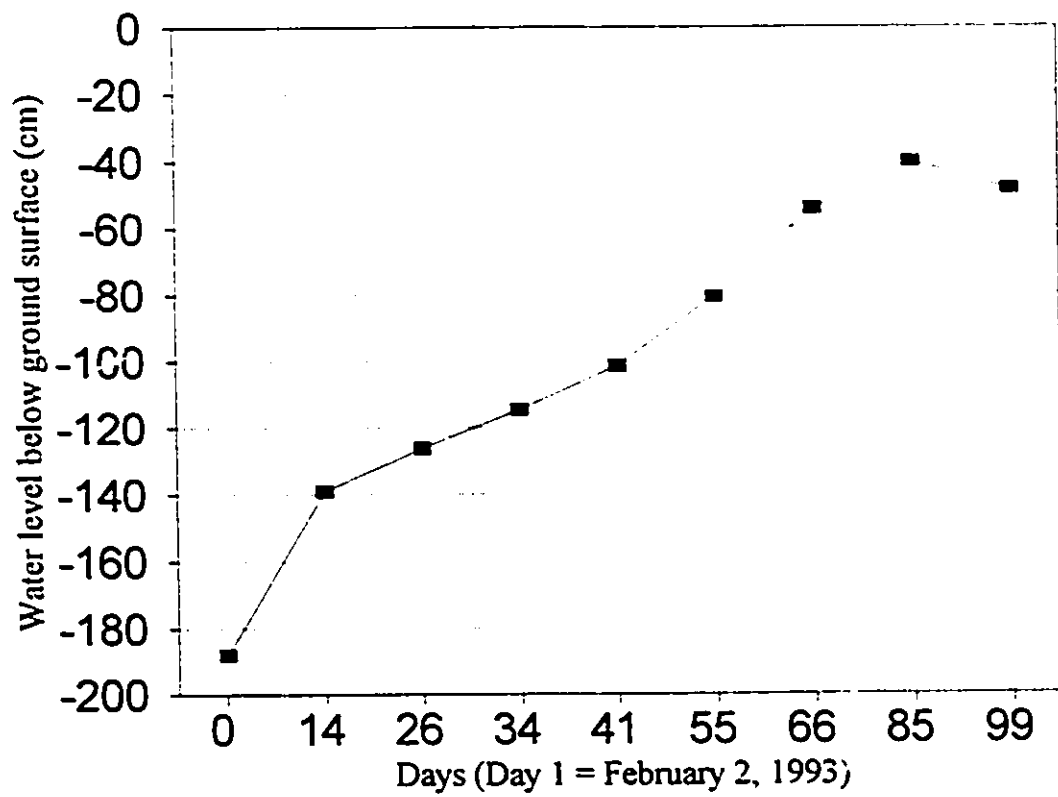


Figure 22. Average recovery data for the Frigin piezometers from February 2 to May 10, 1993.

5.4 Single Well Tests

The hydraulic conductivity values calculated from the single well tests indicated a statistical difference between the OSHM and Frigin methods. 'F' tests were conducted between the K values of the OSHM and Frigin methods (Table 2) and there is no difference between the variances of the two types of tests at the 0.1% significance level. The 'F' test was also performed between the different months (of the same installation method) to determine if the standard deviations of the tests changed based on the time the test was conducted. There was no difference at the 0.1% significance level in this case either. Figure 23 compares the average K values of the two types of test.

5.4.1 OSHM Test Results

The calculated hydraulic conductivity values for the OSHM method are listed in Table 3. The average (geometric mean) hydraulic conductivity value of the forty eight tests is 2.0×10^{-6} m/sec. The values ranged (excluding outliers) from 1.2×10^{-5} to 5.5×10^{-7} m/sec. The outliers were determined using Chauvernet's criterion (Kennedy and Neville, 1986) and are shown in bold italics in the table. Chauvernet's criterion says that an observation in a sample of size n is rejected if it has a deviation from the mean greater than that corresponding to a $1/(2n)$ probability. The probability is calculated on the assumption of a normal distribution, using an estimate of variance on the basis of the sample considered (Kennedy and Neville, 1986). These K values are two orders of magnitude greater than

Table 2. 'F' test results.

OSHM = R	R1:F1	R2:F2	R3:F3
Frigin = F	0.00062	0.58279	0.00045
Junc = 1	>10%	>10%	>10%
July = 2			
August = 3			
	R1:R2	R1:R3	R2:R3
	0.00125	0.03624	0.17973
	>10%	>10%	>10%
	F1:F2	F1:F3	F2:F3
	0.86152	0.04456	0.06505
	>10%	>10%	>10%

Probability given is for the variance in K of the two series being the same.

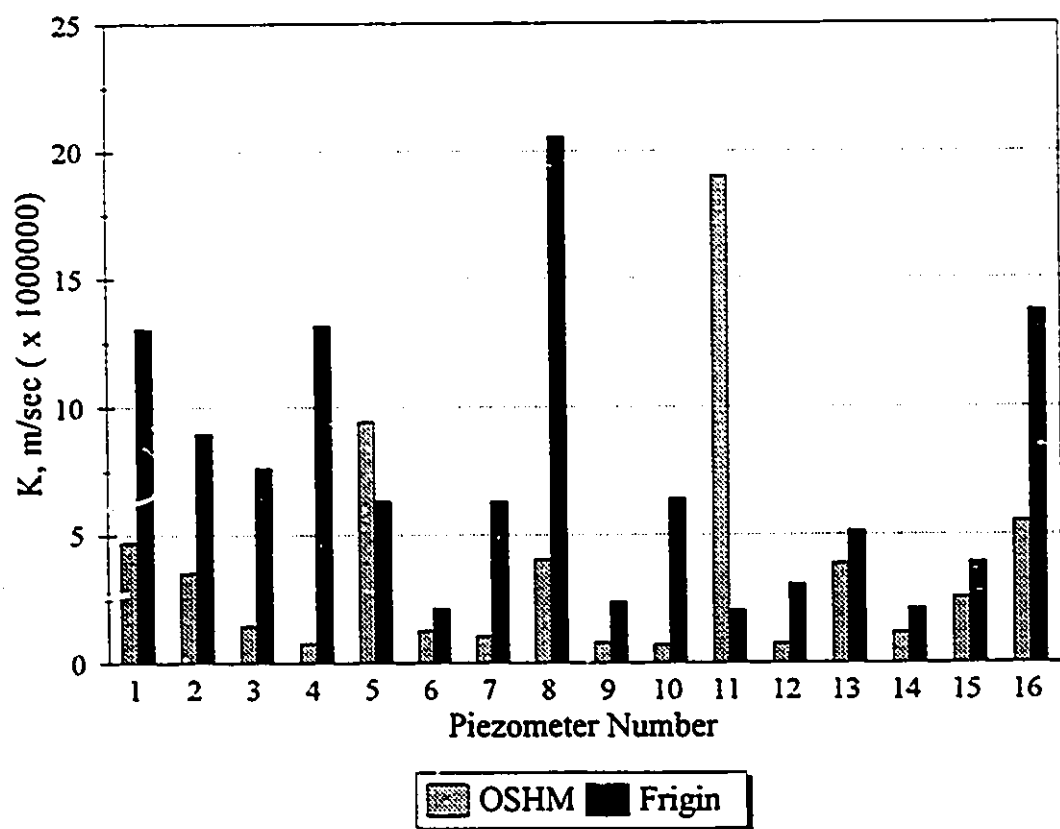


Figure 23. Histogram of average K values.

Table 3. K values for the OSHM series (m/sec).

Piezometer Number	June	July	August
R1	1.4E-06	2.2E-06	1.0E-05
R2	1.3E-06	2.1E-06	7.2E-06
R3	1.3E-06	9.6E-07	2.1E-06
R4	5.5E-07	5.9E-07	1.2E-06
R5	<i>1.2E-05</i>	8.8E-06	7.4E-06
R6	8.2E-07	8.2E-07	2.1E-06
R7	8.2E-07	7.8E-07	1.6E-06
R8	1.6E-06	2.3E-06	8.2E-06
R9	6.0E-07	5.6E-07	1.3E-06
R10	6.2E-07	5.9E-07	9.1E-07
R11	<i>1.4E-05</i>	<i>1.7E-05</i>	<i>2.6E-05</i>
R12	6.0E-07	5.6E-07	1.1E-06
R13	1.5E-06	1.5E-06	8.7E-06
R14	7.1E-07	1.1E-06	1.8E-06
R15	1.8E-06	1.7E-06	4.3E-06
R16	4.6E-06	4.0E-06	8.0E-06

the values previously obtained by D'Astous et al. (1989) in Sarnia, Ontario at the same depth using the same method. Of the piezometers installed using the OSHM method, 69% had K values in the order of 10^{-6} m/sec or greater.

5.4.2 Frigid Insertion Test Results

The calculated hydraulic conductivity values for this test are listed in Table 4. Outliers were found using Cauvenet's criterion and are in bold italics. The average K value (geometric mean) of the forty eight tests is 4.8×10^{-6} m/sec. The values ranged from 2.0×10^{-5} to 6.0×10^{-7} m/sec. Of the piezometers installed using the Frigin method, 90% had K values in the order of 10^{-6} m/sec or greater.

5.4.3 Statistical Analyses Of The Single Well Tests

This section presents the results of the statistical analyses. 't-test' and 'paired t-test' were performed on the data from the field study. The three slug tests were also compared to show the effects of seasonal fluctuations in the groundwater level.

5.4.3.1 "t-test"

Two-sample "t-tests" were conducted on the data. This test compares the means of two random small sized samples (Kennedy and Neville, 1986). It shows if there is a significant difference between the two means. One-sample tests could not be conducted because a population mean was unavailable.

The two geometric means compared in each of the two-sample tests were the means of the hydraulic conductivities (from the single well tests conducted at the same time) calculated for the OSHM series and the Frigin series. Four two-sample tests were performed, one for each set of slug tests (June, July and August 1993) and one that

Table 4. K values for the Frigin series (m/sec).

Piezometer Number	June	July	August
F1	9.8E-06	1.3E-05	3.7E-05
F2	3.5E-06	1.2E-05	1.2E-05
F3	1.4E-06	1.1E-05	1.1E-05
F4	1.4E-05	1.4E-05	1.2E-05
F5	1.6E-06	4.9E-06	1.3E-05
F6	8.3E-07	1.4E-06	4.2E-06
F7	8.3E-07	6.8E-06	1.1E-05
F8	2.1E-05	2.0E-05	2.0E-05
F9	7.1E-07	4.0E-06	2.5E-06
F10	7.0E-06	3.1E-06	9.3E-06
F11	7.9E-07	1.5E-06	3.9E-06
F12	6.0E-07	1.5E-06	7.2E-06
F13	2.0E-06	4.7E-06	8.9E-06
F14	1.2E-06	1.5E-06	3.7E-06
F15	2.3E-06	3.2E-06	6.3E-06
F16	1.3E-05	1.5E-05	1.4E-05

compared the geometric mean of the K values from all the slug tests.

The results are given in Table 5. The combined test results give only a 2% chance that the geometric means of the K values for the different slug tests are the same. The slug tests conducted in June, July and August show a 3%, 2% and >10% chance, respectively, of having the same means.

5.4.3.2 "Paired sample t-test"

The "paired sample t-test" is a comparison between two samples drawn from one source and then subjected to two different treatments whose effects are being studied (Kennedy and Neville, 1986). The "paired sample t-test" was used to compare the hydraulic conductivities of adjacent piezometers since the OSHM and Frigin methods of piezometer installation were alternated down the line of piezometers. Also, because the two piezometer series alternate in a straight line, two sets of the "paired sample t-test" were run. The first set of tests were run against "like" numbered piezometers, i.e. the first piezometer installed by the Frigin method against the first piezometer installed by the OSHM method, the second Frigin piezometer against the second OSHM piezometer, etc. The second set of tests were run against "unlike" numbered piezometers, i.e. with the second piezometer in the Frigin series against the first piezometer in the OSHM series, the third Frigin piezometer against the second OSHM piezometer, etc. This order was chosen because the first OSHM piezometer is flanked by the first two Frigin piezometers.

The results of these tests are given in Table 6. Table 6 indicates a strong difference between K values from the two installation techniques. The probability of there being a difference between the two methods of well installation is greater than 99.9%.

Table 5. Two sample test results.

OSHM n : Frigin n			
June	July	August	Combined
~3%	~2%	>10%	~2%

Probability given is for the K values of the two series being the same.

Table 6. Results of the "paired sample t-test".

OSHM n : Frigin n		
June	July	August
<0.1%	<0.1%	<0.1%

OSHM n : Frigin n+1		
June	July	August
<0.1%	<0.1%	<0.1%

Probability given is for the K values of the two series being the same.

This is true for all of the "paired t-tests" conducted.

5.4.3.3 Same Well

Two-sample "t-tests" and "paired sample t-tests" were also conducted between the same piezometers during different slug tests, i.e. testing the K values of the OSHM piezometers from the slug test conducted in June against the K values of the OSHM piezometers from the slug test conducted in July. The statistical tests were conducted on the data from both the OSHM piezometers and the Frigin piezometers. The pairing of the tests was June vs. July, June vs. August, and July vs. August.

These tests would indicate if there was a difference in the hydraulic conductivity values based on the time of year the slug test was run. The factors affected by the time of year include the height of the water table, temperature of the water, and soil moisture. These factors would affect fracture aperture. These factors were a concern as the tests were being run because differences were readily apparent in the speed of recovery with time.

The results of the two-sample "t-test" (Table 7) indicate that there was no difference between the K values determined in June and July for either the OSHM or the Frigin method of piezometer installation. For the OSHM method, there is only a 9% chance that the August K values were statistically the same as the June K values and only a 10% chance that the August K values were the same as the July K values. For the Frigin method, there is no difference between the K values in any month.

The comparisons between the various months (same pairing as above) using the "paired sample t-test" method for both the OSHM and Frigin installation methods, indicate

Table 7. Two sample test results, same piezometer.

OSHM

June : July	June : Aug	July : August
>10%	~9%	~10%

Frigin

June : July	June : Aug	July : August
>10%	>10%	>10%

Probability given is for the K values of the two series being the same.

that there is a much greater chance than 99.9% of there being a difference in the K values based on the time the test was run (Table 8). The "paired t-test" provides strong evidence that the time of year a slug test is conducted affects the calculated hydraulic conductivity in weathered clay tills.

5.4.3.4 Temperature Effects

The temperature of the water has an effect on the K values calculated. This is due to the viscosity changes of water at different temperatures. At the beginning of June, the 20 year average temperature at 100 and 150 cm was 12.5°C and 11.2°C, respectively. The highest 20 year average temperatures were recorded at the beginning of September. The temperature at the same depths were 18.8°C and 17.6°C, respectively. The increase in temperature causes a corresponding decrease in the viscosity of water. The viscosity decreases by 15% (Weast, 1973). Since viscosity is in the denominator, the K value calculated from the water at higher temperature may be up to 17% higher than that calculated at the lower temperature.

The effects of temperature were not taken into account when calculating K for this project. This was because the in situ K was what was important. The temperature correction should only be used to normalize the K values.

5.4.3.5 Centred and Sorted

A set of two-sample "t-tests" and "paired sample t-tests" was conducted on the hydraulic conductivity values after the three highest and three lowest values of each data set were removed. There were two reasons for this statistical test.

First, if any of the piezometers intersected "large" fractures, the Frigin method

probably would not have made a difference in the response of the piezometers. This assumption is based on data that show that the OSHM method of piezometer installation is in itself a good method to prevent smearing of clay (D'Astous et al., 1989). Fractures of sufficient aperture would probably not be smeared closed using this technique. In this case, the highest K values would be the same using either technique. If they were removed, the differences between the two methods would be more apparent.

Second, the lowest K values were removed because some of the piezometers may not have intersected any fractures regardless of the insertion method. In this case, the method used to install the piezometers would make no difference. Removing the lowest K values would again amplify the differences between the OSHM and Frigin methods of piezometer installation.

The remaining hydraulic conductivity values from all the tests are shown in Table 9. The piezometers removed from the data set were not the same from test to test. For example, the K value for R5 was removed in June but not August. In June, the K value from this piezometer was one of the three highest for that slug test, but it was not one of the three highest values for the August slug test.

Outliers were included before the K values were removed. The outliers were not errors in the data, just piezometers that had low K values. The outliers were included because the maximum number of data points was wanted. The assumptions made for removing the six data points in each series took into account the factors that made the outliers outliers.

Table 8. "Paired sample t-test" results, same piezometer.

OSHM		
June : July	June : August	July : August
<0.1%	<0.1%	<0.1%
Frigin		
June : July	June : August	July : August
<0.1%	<0.1%	~1.5%

Probability given is for the K values of the two series being the same.

Table 9. K values for the centred and sorted data.

OSHM			Frigin		
June	July	August	June	July	August
6.2E-07	5.9E-07	1.3E-06	8.3E-07	1.5E-06	4.2E-06
7.1E-07	7.8E-07	1.6E-06	8.3E-07	3.1E-06	6.3E-06
8.2E-07	8.2E-07	1.8E-06	1.2E-06	3.2E-06	7.2E-06
8.2E-07	9.6E-07	2.1E-06	1.4E-06	4.0E-06	8.9E-06
1.3E-06	1.1E-06	2.1E-06	1.6E-06	4.7E-06	9.3E-06
1.3E-06	1.5E-06	4.3E-06	2.0E-06	4.9E-06	1.1E-05
1.4E-06	1.7E-06	7.2E-06	2.3E-06	6.8E-06	1.1E-05
1.5E-06	2.1E-06	7.4E-06	3.5E-06	1.1E-05	1.2E-05
1.6E-06	2.2E-06	8.0E-06	7.0E-06	1.2E-05	1.2E-05
1.8E-06	2.3E-06	8.2E-06	9.8E-06	1.3E-05	1.3E-05

The differences between the tests might indicate the extent to which the fracture networks are interconnected. A piezometer that drained faster in a later test might indicate a larger fracture network. Another reason may be the extent of the interconnection at different depths. A higher water table may result in a more hydraulically connected fracture system, fractures might become more isolated as water levels dropped. This would cause piezometers to respond more slowly later in the summer.

The ten remaining piezometers in each data set were sorted in ascending order. The sorting was done on the assumption that piezometers with higher hydraulic conductivities intersected more fractures or larger fracture networks and that the piezometers with lower K values intersected fewer fractures or smaller fracture networks. The sorted K values would then compare piezometers that intersect a similar number of fractures.

Two-sample "t-tests" and "paired sample t-tests" were performed. The statistical analysis indicates that there is a significant difference in the two piezometer installation techniques (Table 10). The two-sample test, which compared the OSHM and Frigin methods for each of the three months gave the probability of the two techniques giving the same K values as >10%, 2% and 0.5% for June, July and August, respectively. Taking the geometric mean of all the K values for the three months gave a probability of 0.5% that the two techniques have the same K values. The "paired sample t-test" results were even more conclusive, all giving a probability of less than 0.1% of being the same (Table 11).

Table 10. Two sample test results, centred and sorted.

OSHM : Frigin		
June	July	August
>10%	~2%	~0.5%
All Months		
~0.5%		

Table 11. "Paired sample t-test" results, centred and sorted

OSHM : Frigin		
June	July	August
<0.1%	<0.1%	<0.1%

Probability given is for the K values of the two series being the same.

Two-sample "t-tests" and "paired sample t-tests" were also performed between the same piezometers to determine the effect the time of year the test was run had on the hydraulic conductivity (as described in the previous section). The results of the "paired t-test" test are given in Table 12. The "paired t-test" showed a less than 0.1% probability that the hydraulic conductivity values obtained in June were from the same population as those obtained in July or August. Likewise, the July K values had a less than 0.1% chance of being from the same population as the August values.

The results of the two-sample "t-test" are given in Table 13. They show that there was little difference between the months of June and July for either test. August again proved to be much different than either June or July. The probability that the OSHM method's K values from June and July are from the same population as August's are only 4% and 6%, respectively. There is less than a 0.1% chance that the Frigin method's June K values are from the same population as August's and there is a 4% chance that July's K values are the same as August's.

5.4.4 Single Well Test Discussion

The hydraulic conductivity values obtained in this project are on the order of 10^{-6} m/sec. This range of values is very high for the soil type. A study by McKay and Cherry (1992) included preliminary results (derived from seepage collectors in tracer test trenches) where hydraulic conductivities of this order of magnitude were also obtained in weathered clay near Sarnia, Ontario. D'Astous et al. (1989) used the OSHM method of piezometer installation in weathered clays near Sarnia at the same depth and obtained results on the order of 10^{-8} m/sec. Hendry (1982) found a fissured glacial till in Alberta

Table 12. "Paired sample t-test" results, same piezometer, centred and sorted.

OSHM		
June : July	June : August	July : August
<0.1%	<0.1%	<0.1%
Frigin		
June : July	June : August	July : August
<0.1%	<0.1%	~1.5%

Table 13. Two sample test results, same piezometer, centred and sorted.

OSHM		
June : July	June : August	July : August
>10%	~4%	~6%
Frigin		
June : July	June : August	July : August
~10%	<0.1%	~4%

Probability given is for the K values of the two series being the same.

had K values on the order of 10^{-7} m/sec. The difference between his average K values and those found in this study may be in part due to the condition of the weathered clay (different locations).

Chiasson (1992) reported that the fracture density of the weathered clay at the Whelan Farm Site was greater than that at his other test site, about 6 km away. This other site is located near Essex County Landfill No.3 and is referred to as the Puce Road Site. The Puce Road site more closely resembles the D'Astous et al. (1989) site near Sarnia. The land use and the ground cover are the same at Chiasson's (1992) Puce Road site and D'Astous et al. (1989) site near Sarnia.

The greater fracture density at the Whelan Farm site may have been brought about by the land use. The Whelan Farm Site is an agricultural site, while the Puce Road site is not. Chiasson (1992) detected root holes and fractures at greater depths at the Whelan site than at his other site. The presence of crops would have increased the evapotranspiration rate at the Whelan site, which would have in turn increased the dessication rate of the clay, leading to a denser and deeper fracture network. The Whelan Farm site also has a network of tile drains, which would facilitate drainage. The tile drains are set 60 cm below the ground surface. There are two tile drains in each of the Whelan Farm site test plots and they run the length of the plot. The larger fracture network and the greater fracture density may account for the higher bulk hydraulic conductivity values.

The time of year that the slug tests were conducted had a large affect on the hydraulic conductivity values. Increasing desiccation of the clay towards the end of the summer increased the calculated hydraulic conductivity of the piezometers to such an

extent that statistically they were unrelated to the K values from the slug tests conducted in the early summer. This effect raises a question of which values are correct.

Other questions come to mind. Does the time of year the piezometer is installed (soil moisture) have any affect on the smearing of clay? Does recovery of a piezometer depend on the time of year?

5.5 Error Analysis

In the lab experiments, the soil cylinder was not fully repacked between the second and third run where the cylinder was not frozen before drilling, and between the first and second run when the cylinder was frozen. The cylinder was repacked from the fracture upward. It was not fully repacked because it was difficult to position the filter paper on the sides of the cylinder while packing the clay. Also, it was decided not to repack the entire column since the lower portion of the cylinder remained intact.

Other factors that may have affected the results of the constant head test were: the density of packing of the clay in the cylinder, not changing the filter paper after every test, and the change in the hydraulic head while the tests were being run. The density of packing may not have been consistent from test to test. The major aim in packing the cylinder was to prevent short-circuiting, not to maintain a uniform density. The filter paper along the sides of the cylinder may have become clogged with clay because they were not changed for every test. The changes in hydraulic head, while minimal in all but one case, may have affected the calculated K values more than anticipated.

In the field study, the piezometers that were installed using the Frigin method were not all installed to the same depth. This occurred because the drill rig could not push the

second Shelby tube through all of the frozen clay. Even a small stone can stop the Shelby tube in the frozen clay. Half of the Frigin piezometers were installed to 205 cm (the same depth as the OSHM piezometers). The depth of the screened section of the remaining Frigin piezometers varied by 15 cm (50 cm in one case). The piezometers installed by the OSHM method were all installed to the same depth.

Another problem that was discovered after the Frigin piezometers were installed was that not all of the soil borings froze properly. While extracting the soil from the large Shelby tubes, there were four and a half borings that were not frozen or fully frozen. In properly frozen soil borings, the hole made by the first Shelby tube remained intact. The hole collapsed in the soil borings where the soil did not freeze sufficiently and in the piezometers installed by the OSHM method. The specific piezometers where this occurred are not known because the large Shelby tubes were not marked with piezometer identification numbers. It was thought no useful information would be obtained from the large Shelby tubes.

Some problems were encountered removing the clay from the smaller Shelby tubes. These problems occurred since there was no apparatus available to remove the soil from the customized tubes. The last of the soil was removed from the Shelby tubes about two weeks after the piezometers were installed and some desiccation may have occurred, skewing the observations toward more fracture occurrence than there actually was.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

Based on both the two-sample test and the "paired sample t-test", the Frigin method gives significantly higher K values than the OSHM method for fractured clay soils. The two-sample test gave a probability of only 2% (combining the K values from the three tests) and the "paired sample t-test" gave a probability of 0.1% (all tests), that the OSHM and Frigin methods give the same results. Considering that the OSHM method is already much better than the conventional methods, the Frigin method should give much more representative K values than conventional installation methods.

This is important because estimates of groundwater and chemical velocity based on these values will be higher than estimates based on values obtained using other methods. For example, taking the geometric mean of the hydraulic conductivity values from this project, the same hydraulic gradient, and the same soil type, the water would travel approximately 49 m and 81 m (using the K values from the OSHM and Frigin methods of piezometer installation, respectively) in one year. The Frigin method predicts groundwater movement to be 165% faster than that predicted by the OSHM method. In cases where obtaining an accurate estimate of K is important, freezing the clay prior to drilling should be considered. The modification to the OSHM method is neither expensive nor time-consuming and it is easy to perform if a method of transporting liquid nitrogen is available.

6.2 Recommendations

In studies such as this, it is always advisable to do more experiments. A larger number of piezometers would have made the statistical results more reliable. In the future,

if any piezometers are installed using this procedure, it would be easy to check if the soil was frozen by looking down the large shelly tube. If the walls of the first borehole are smooth and intact, the soil is frozen.

The depth of the water table and the resulting variations in soil moisture affected the hydraulic conductivity significantly. When testing the hydraulic conductivity of a weathered clay, you must take into account the time of year and precipitation. The seasonal variation of the hydraulic conductivity would be an interesting subject of further research.

Since the Frigid Insertion method of piezometer installation is effective and not very expensive or difficult, it should be used whenever accurate estimations of the hydraulic conductivity are required.

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APPENDIX A

EXAMPLE CALCULATION FOR FREEZING THE CLAY IN THE CONTROL VOLUME

The following calculations show the amount of heat that would need to be absorbed by the ground to freeze the clay surrounding a Shelby hole to the limits set in the text and the time required to do so. The physical properties used are from Incropera and DeWitt (1990) and Weast (1973). Some of the properties are estimates because of the unique conditions of the study.

Equation 1 is used to calculate the amount of heat absorbed by the clay, which is the amount of heat given up by the liquid nitrogen.

$$q = \frac{2\pi Lk(T_1 - T_2)}{\ln\left(\frac{r_2}{r_1}\right)} \quad (1)$$

$$q = [2(\pi)(0.4\text{m})(2.3\text{W/mK})(273\text{K}-77\text{K})]/[\ln(0.175/0.025)] = 582 \text{ Watts}$$

Equation 2 is used to calculate the amount of liquid nitrogen (in grams per second) needed to absorb the heat calculated above.

$$m = \frac{q}{h_{fg}} \quad (2)$$

$$m = 582 \text{ W} / 47.6 \text{ Cal/g} = 3 \text{ g/sec}$$

This value is for perfect efficiency, which is impossible to achieve.

The control volume used causes the Biot number to go to zero, so Equation 3 is not needed in this instance. Equation 4 uses the ratio of temperature differences along with the Heisler Charts to obtain the Fourier number.

$$\theta_o = \frac{T_o - T_s}{T_i - T_s} \quad (4)$$

$$\text{Theta} = [273-77]/[285-77] = 0.94$$

Using Theta and the Heissler Chart for an infinite cylinder gives a value of 0.10 for the Fourier number, which is used with Equation 5 to calculate the time necessary to freeze the clay to the desired parameters.

$$Fo = \frac{\alpha t}{r_o^2} \quad (5)$$

$$0.10 = [(1.46 \times 10^{-6})(\text{sec})]/(0.15)^2 = 1541 \text{ seconds} = 26 \text{ minutes}$$

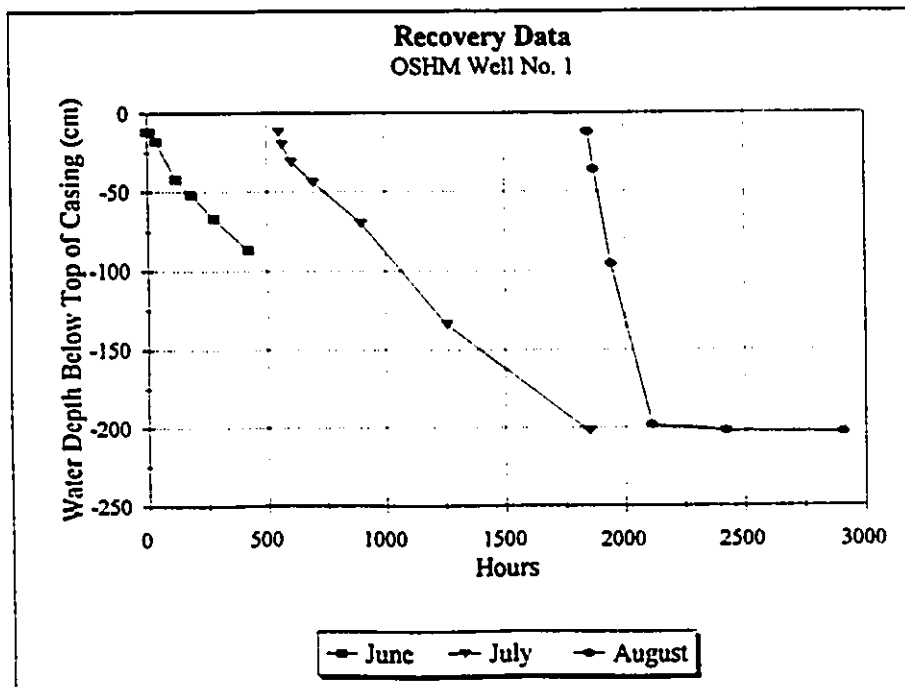
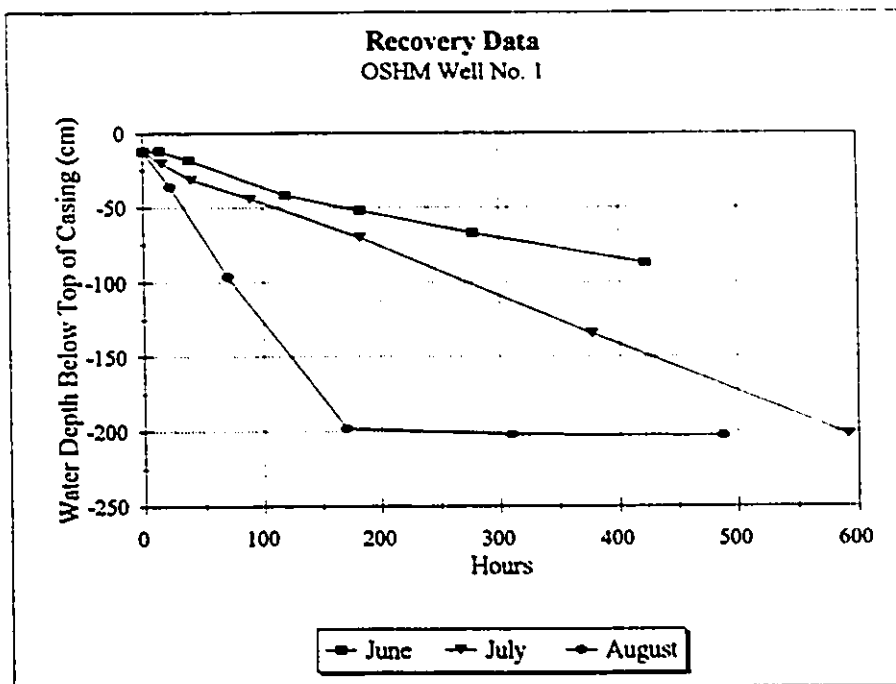
Similar calculations reveal that it takes one minute to freeze the first 3 cm of the clay past the wall of the Shelby hole.

APPENDIX B**RECOVERY DATA (GRAPHICAL) AND HYDROGRAPHS**

The following pages contain two types of graphs for each well. The top graph on each page shows the recovery data for each single well test run. The lower graph shows the cumulative time of all the single-well tests. The lower graph also gives an indication of the water table drop that occurred as the summer progressed. This can be most easily seen on the graphs of wells that showed a rapid response.

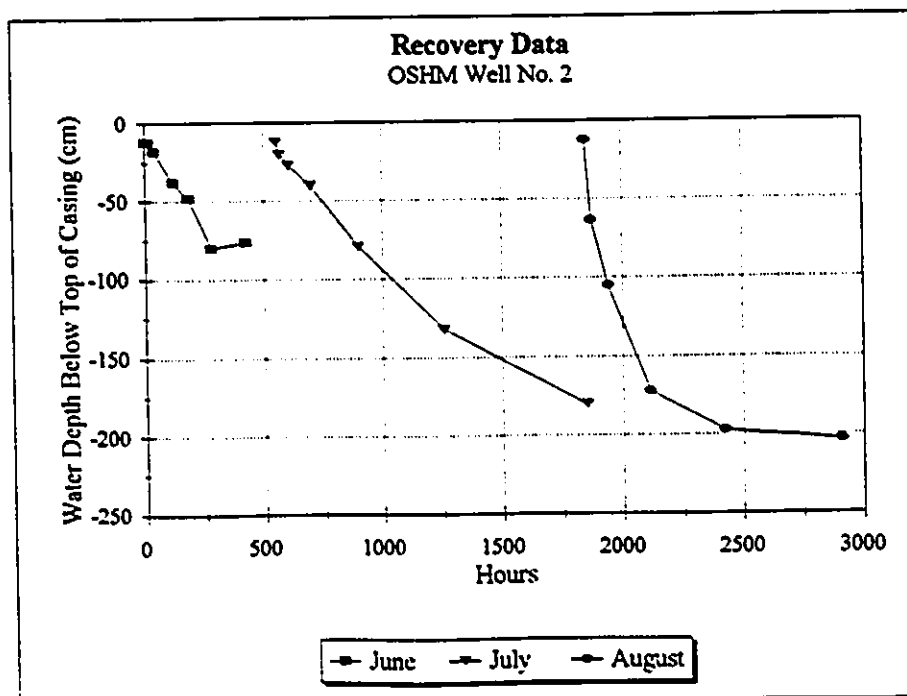
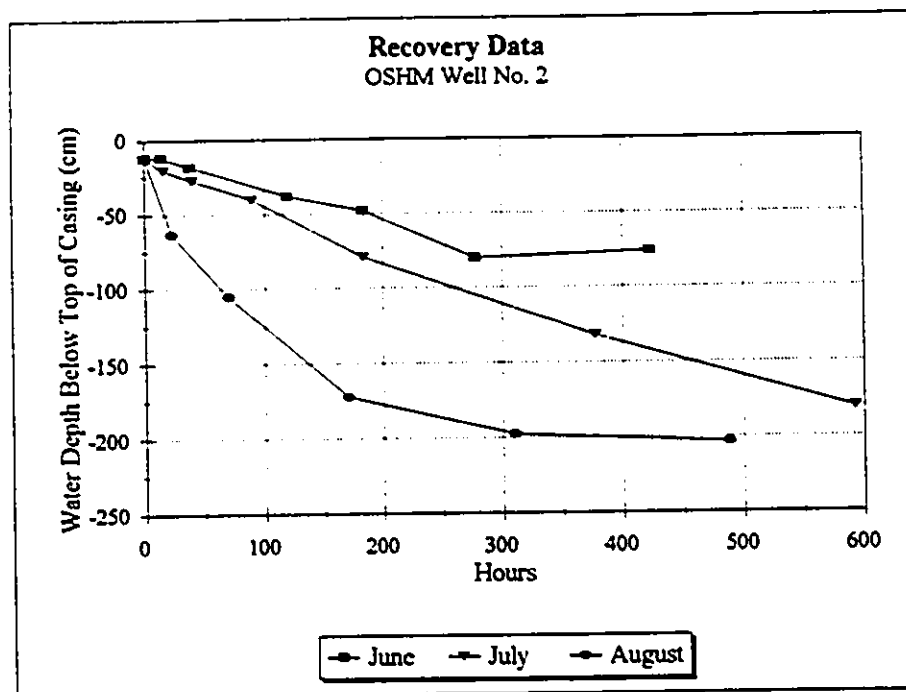
Recovery Data Plots

Individual Tests and Cumulative Time



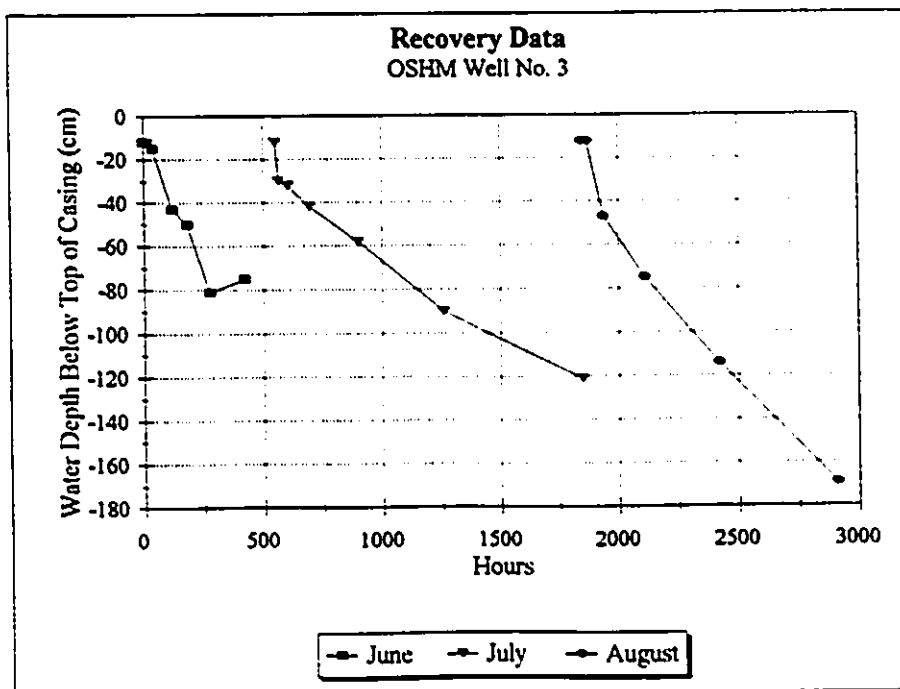
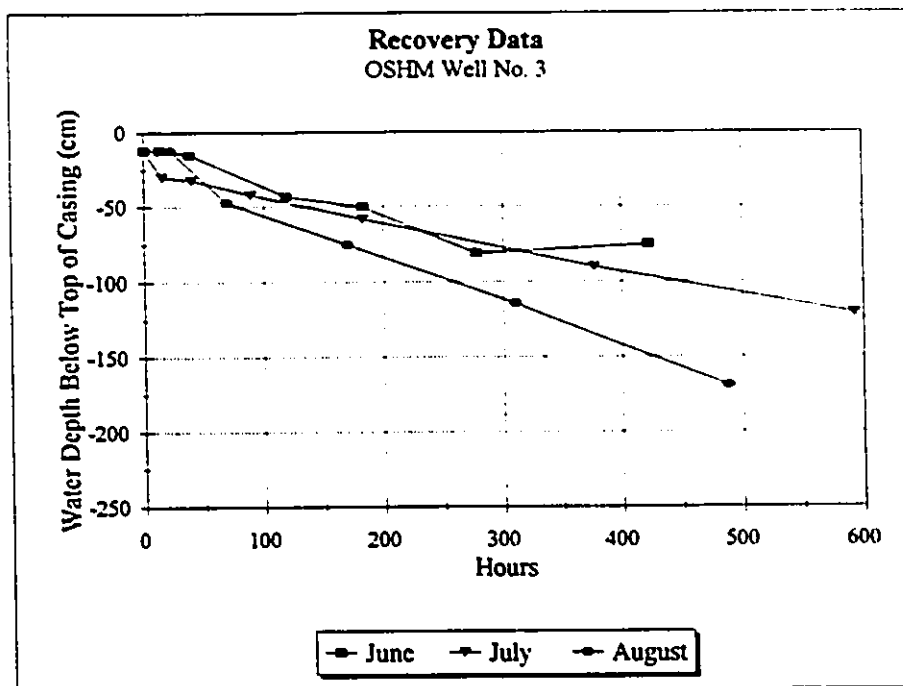
Recovery Data Plots

Individual Tests and Cumulative Time



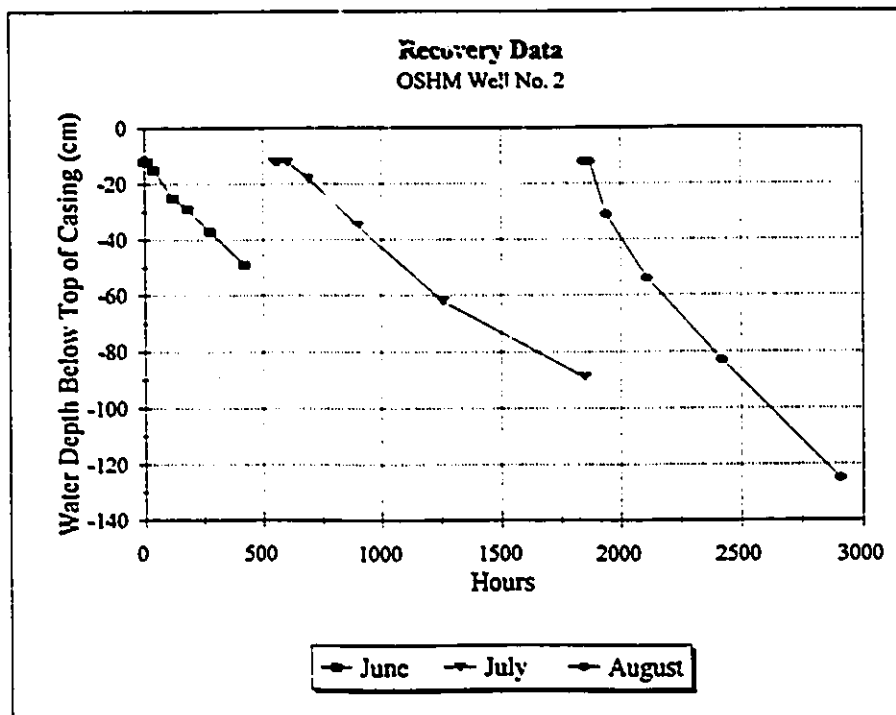
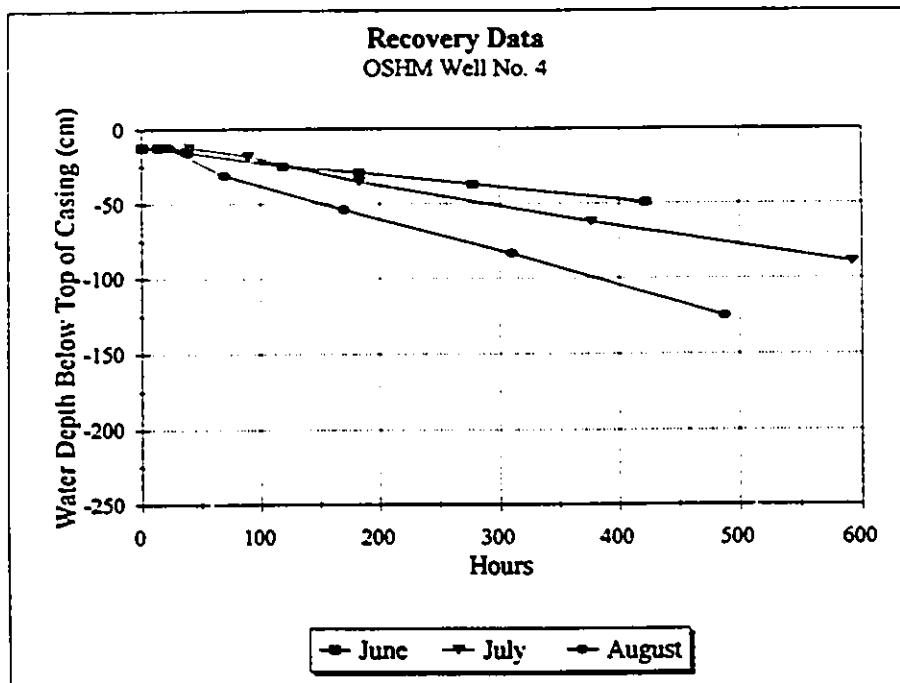
Recovery Data Plots

Individual Tests and Cumulative Time



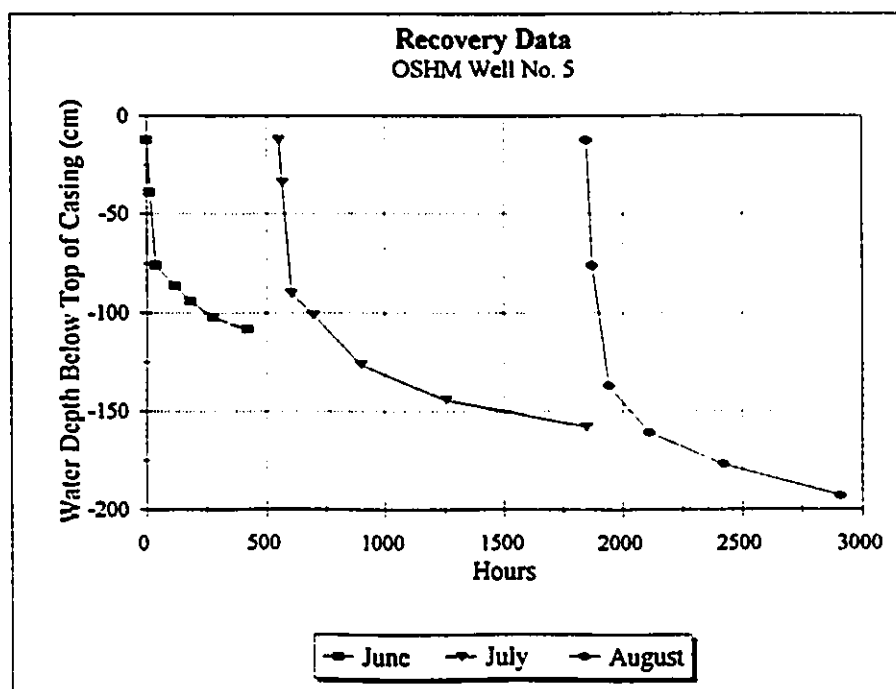
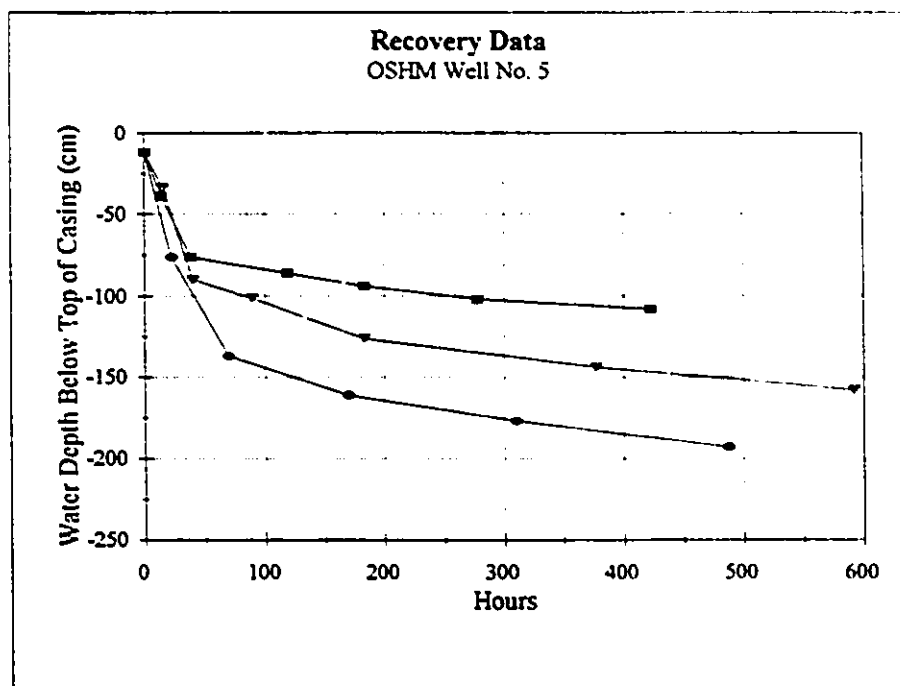
Recovery Data Plots

Individual Tests and Cumulative Time



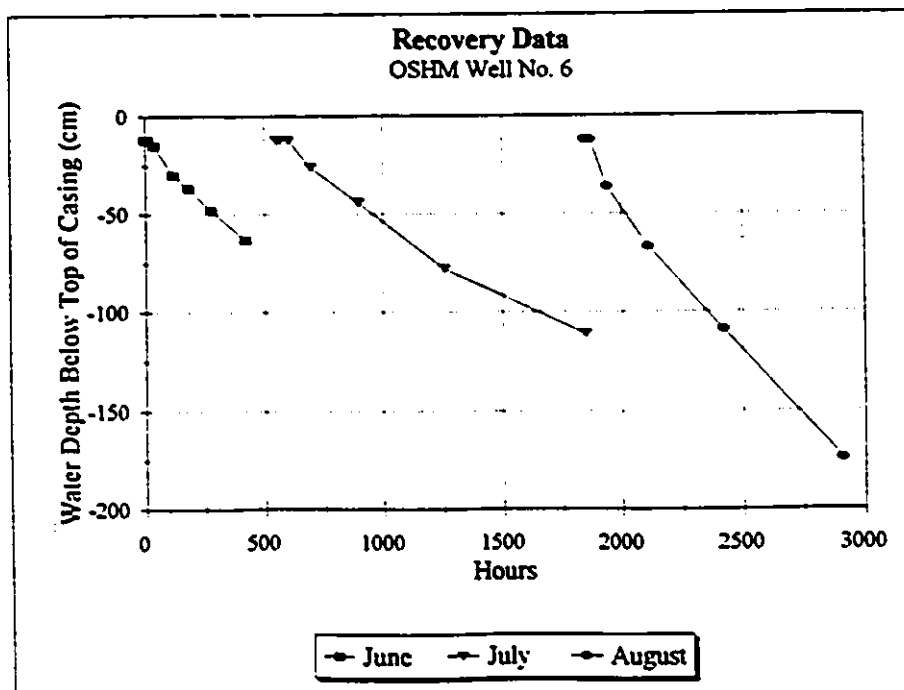
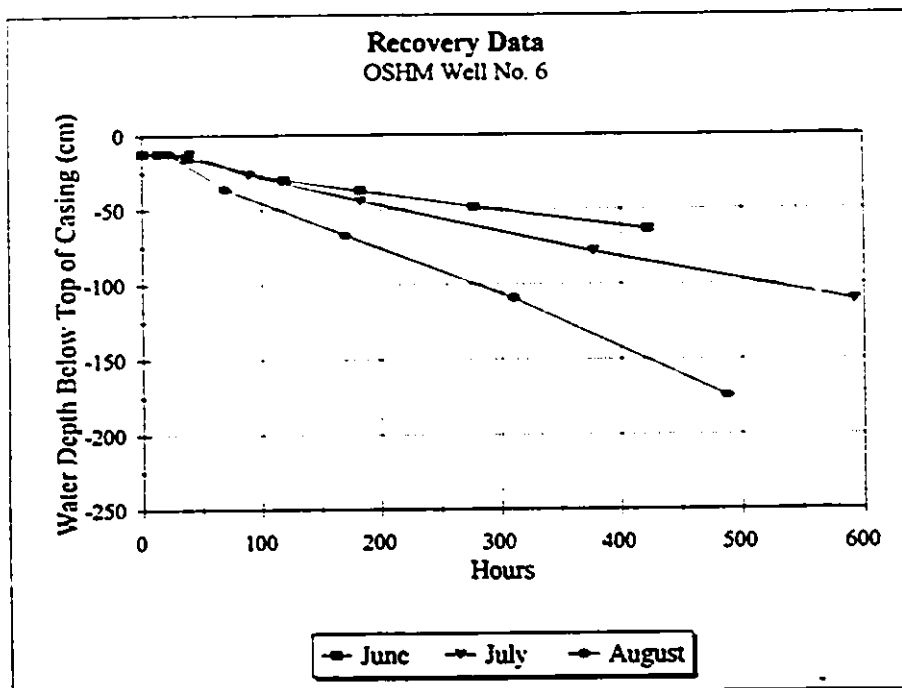
Recovery Data Plots

Individual Tests and Cumulative Time



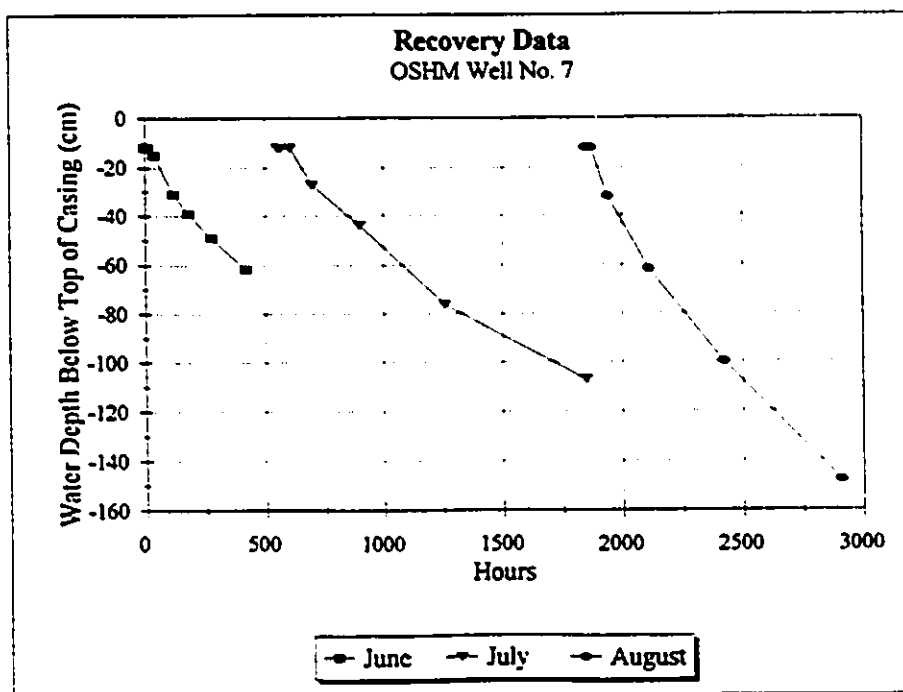
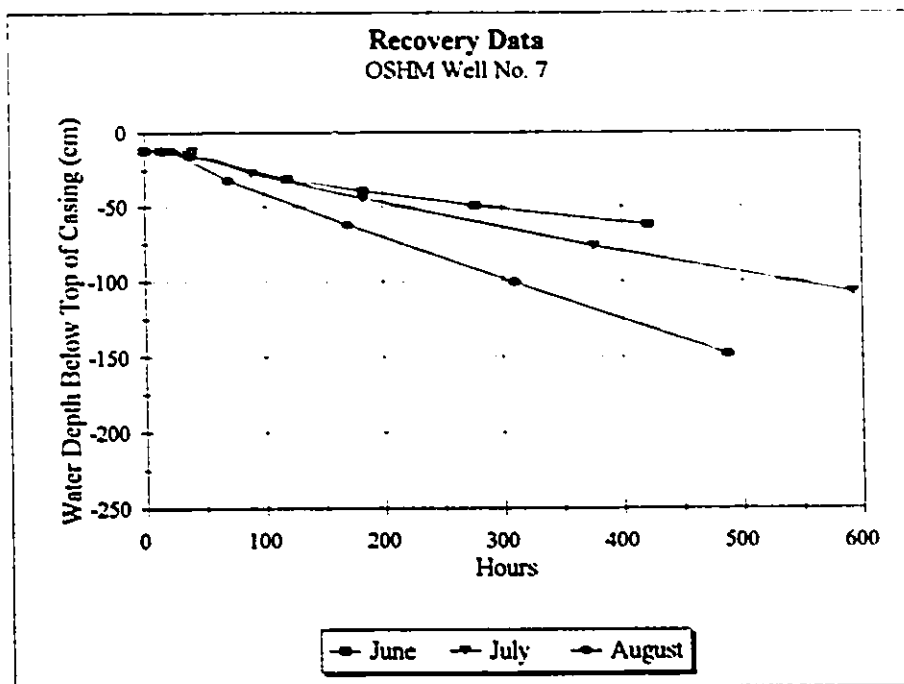
Recovery Data Plots

Individual Tests and Cumulative Time



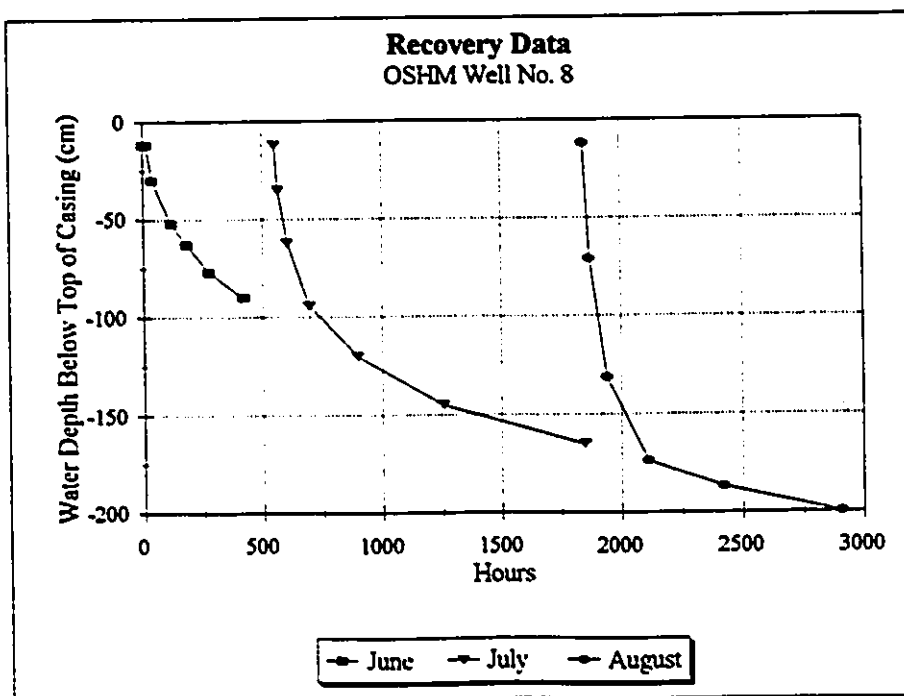
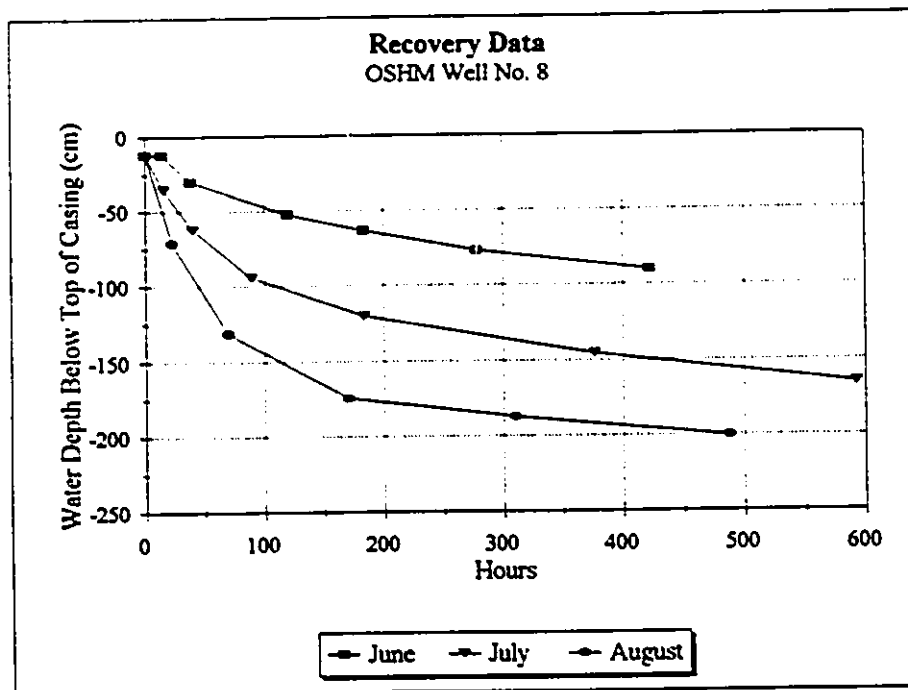
Recovery Data Plots

Individual Tests and Cumulative Time



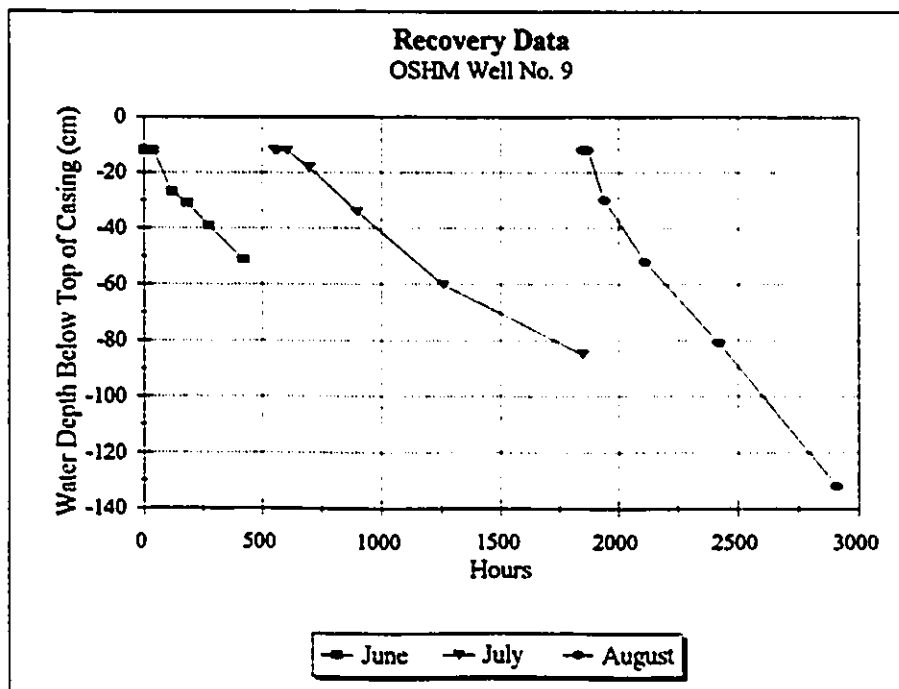
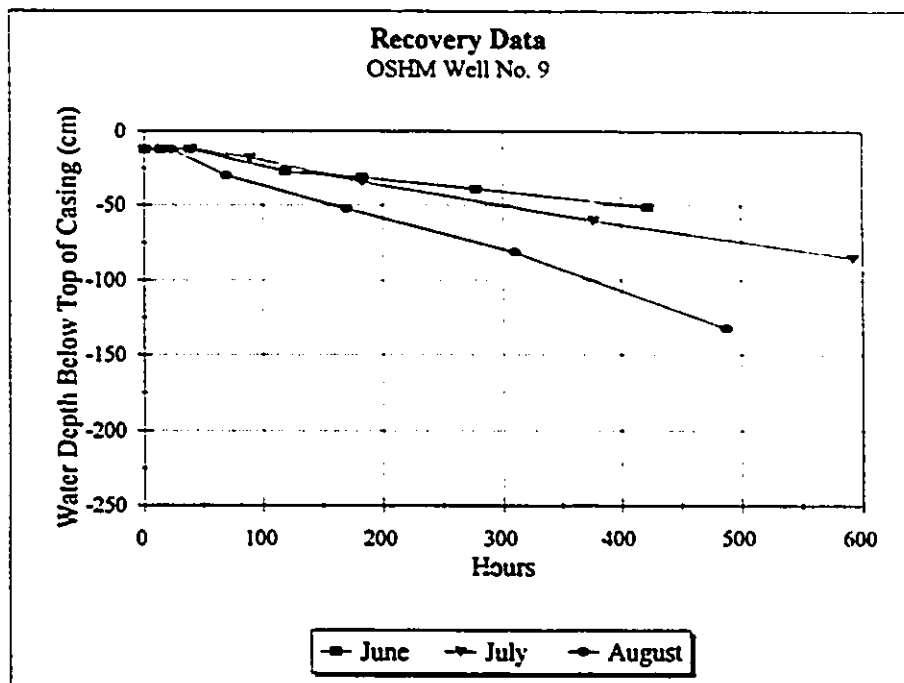
Recovery Data Plots

Individual Tests and Cumulative Time



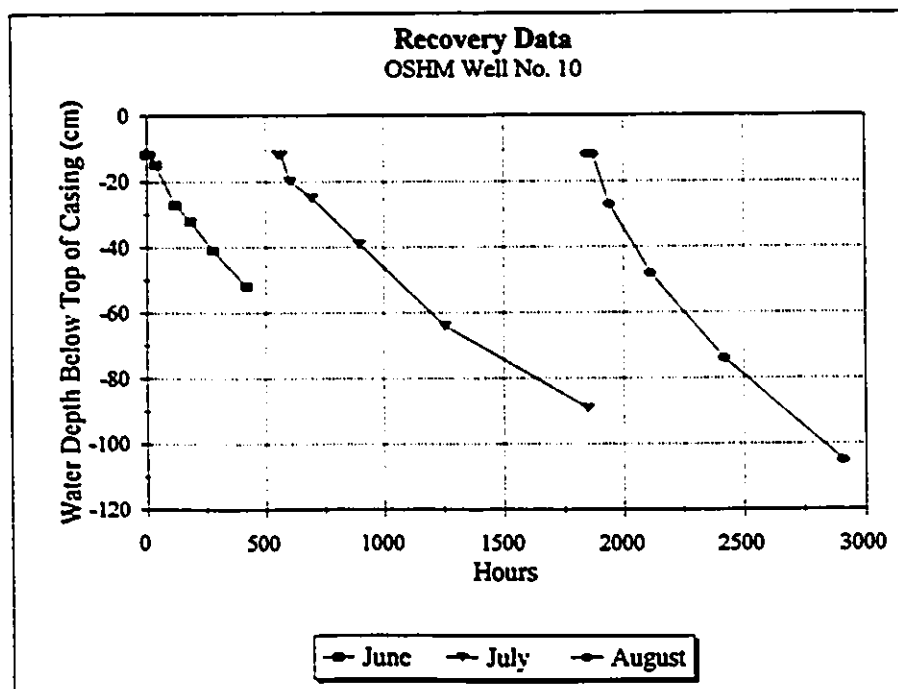
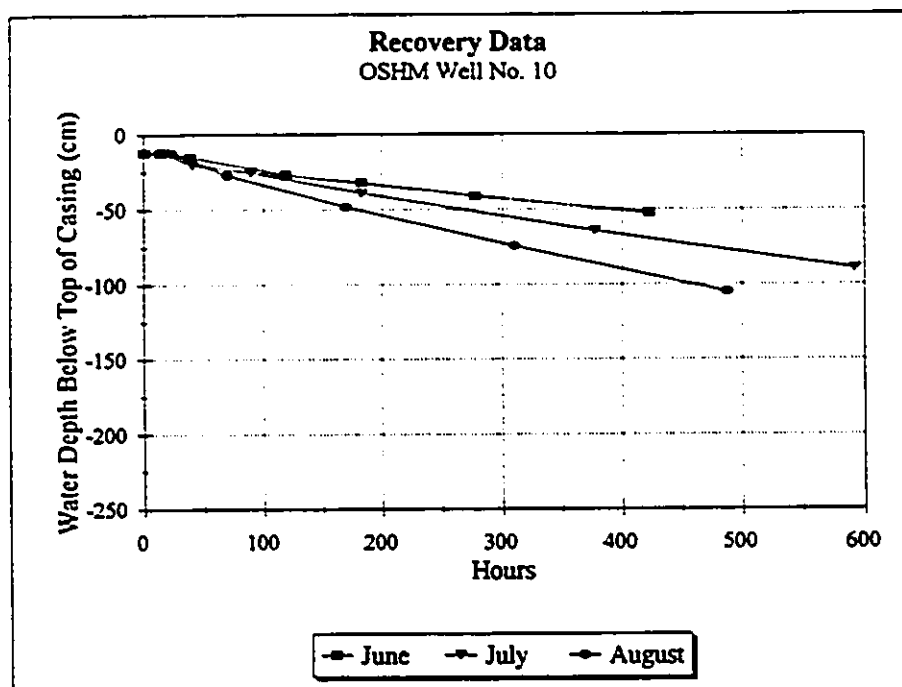
Recovery Data Plots

Individual Tests and Cumulative Time



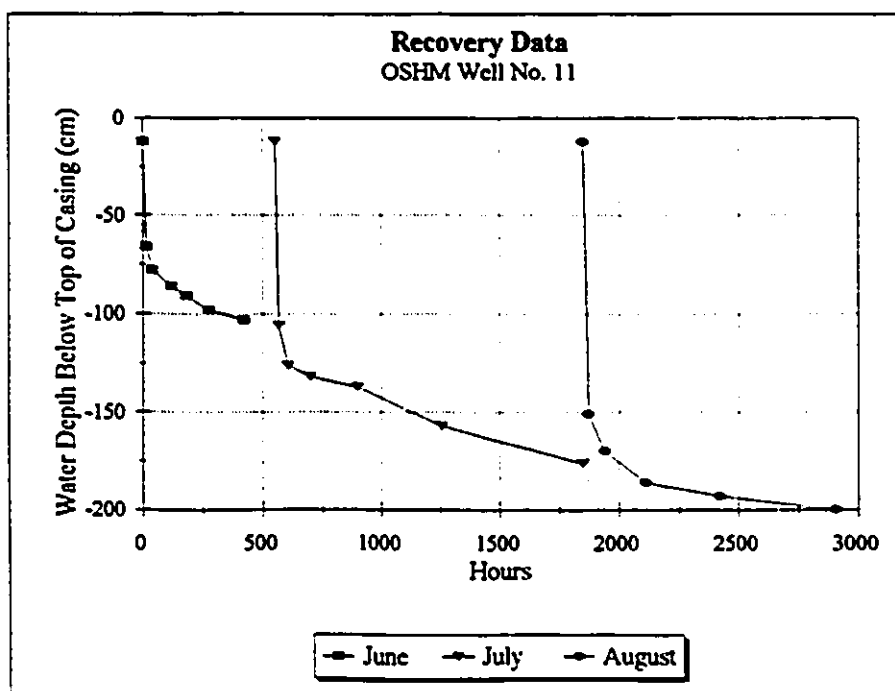
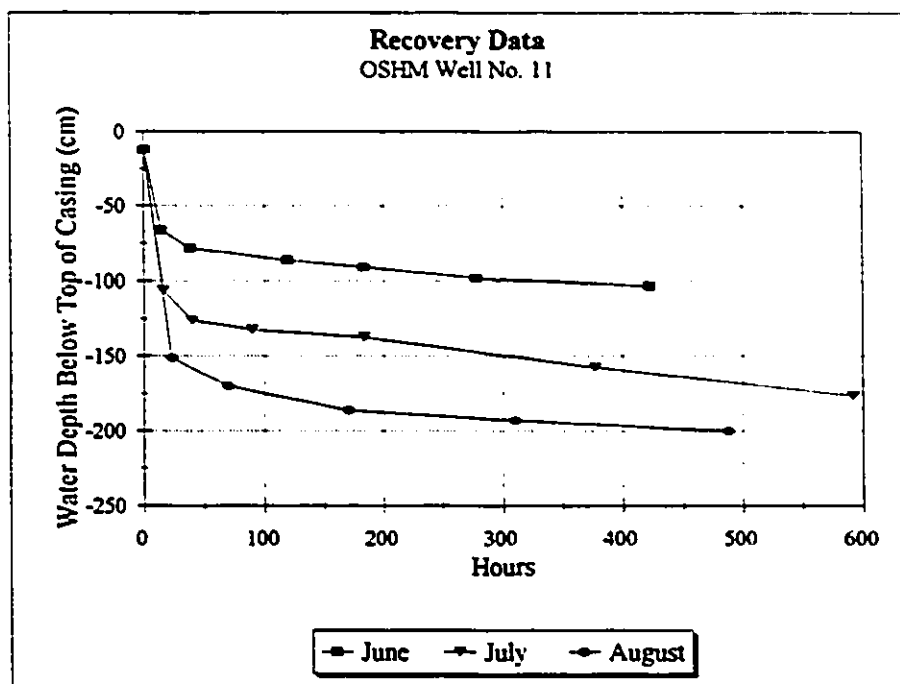
Recovery Data Plots

Individual Tests and Cumulative Time



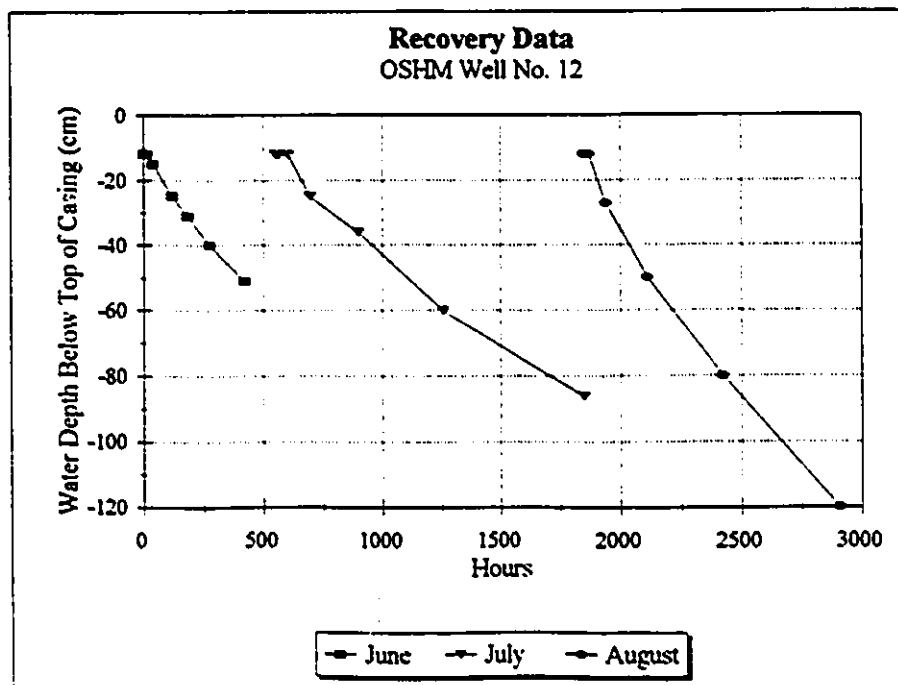
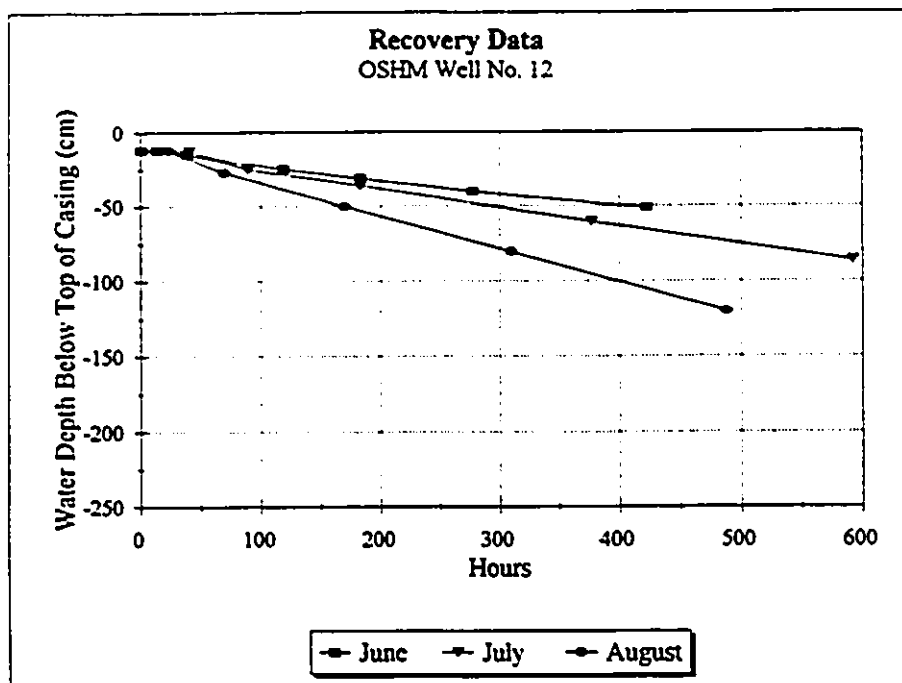
Recovery Data Plots

Individual Tests and Cumulative Time



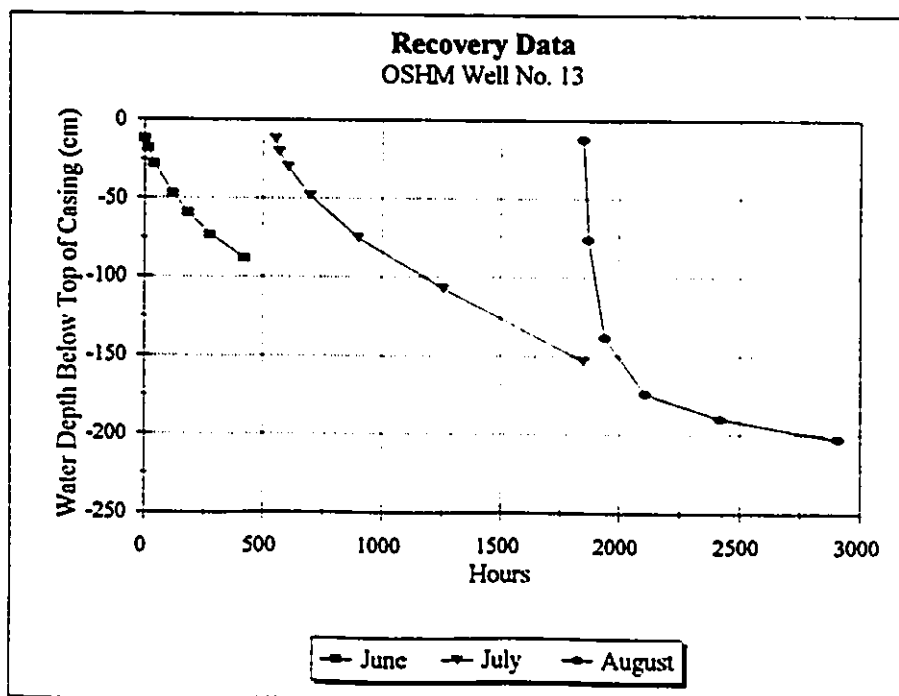
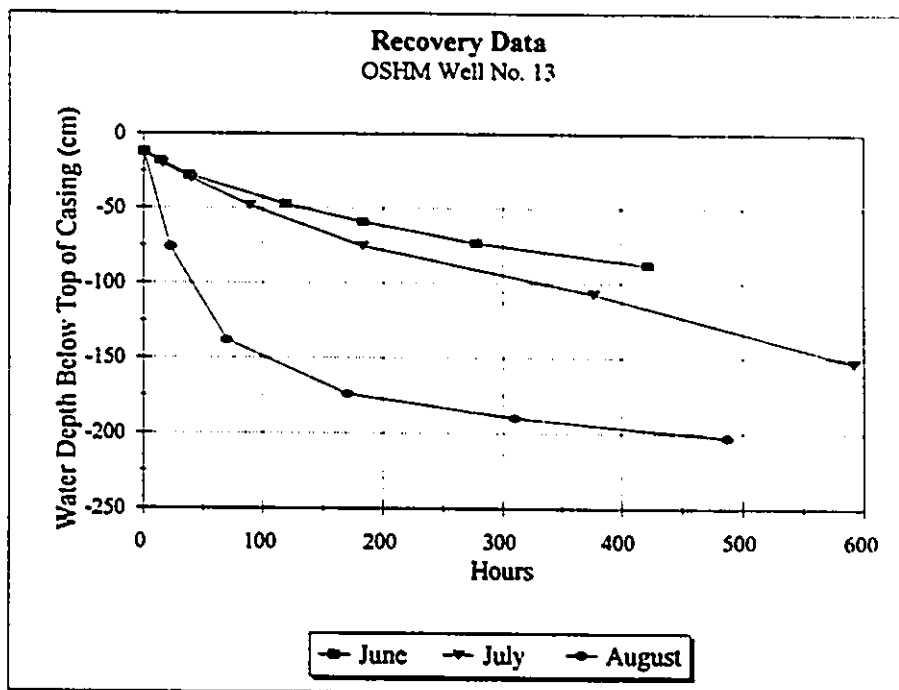
Recovery Data Plots

Individual Tests and Cumulative Time



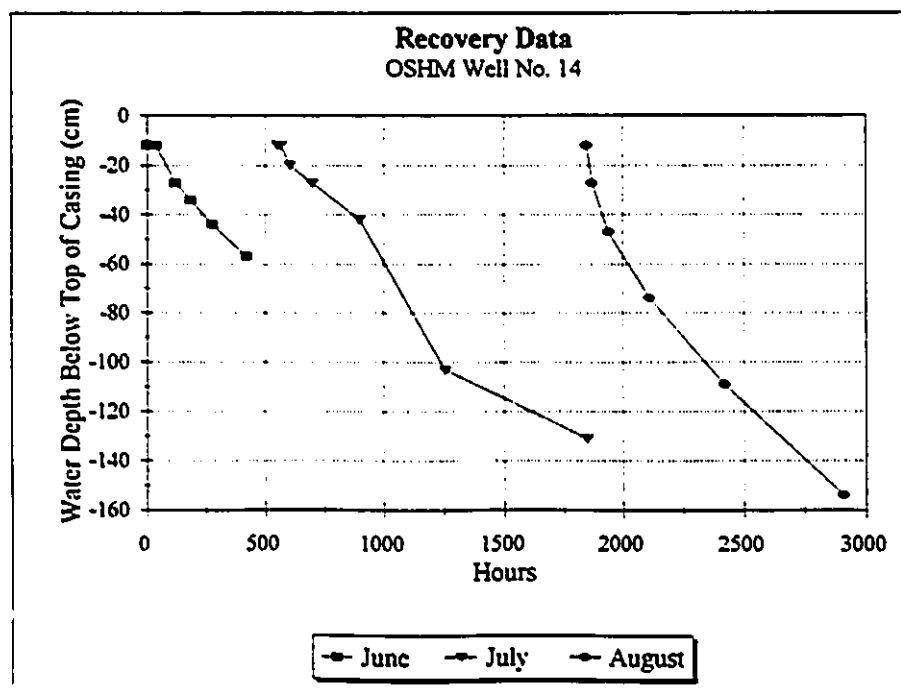
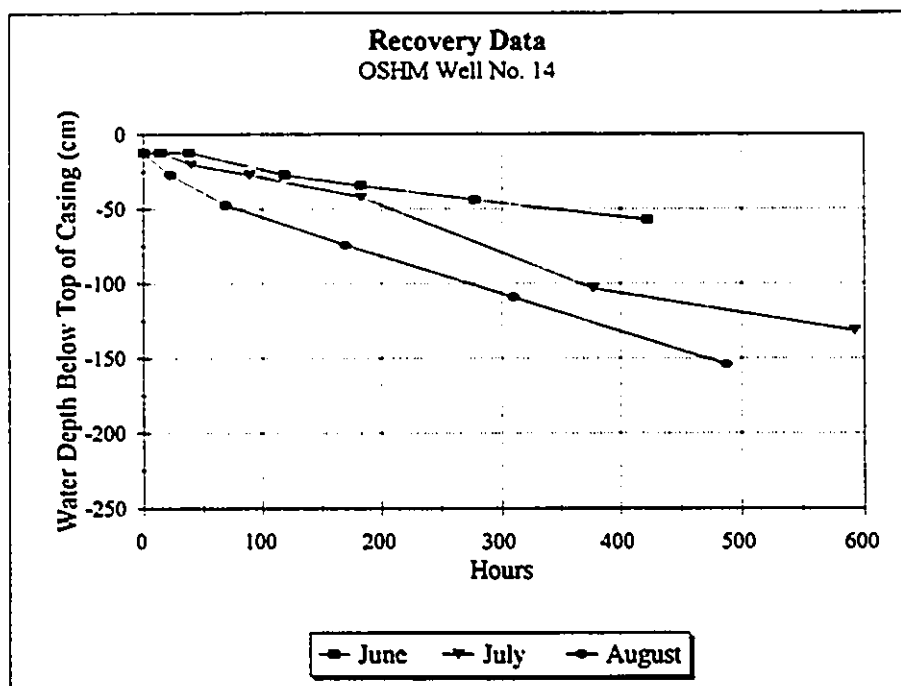
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Individual Tests and Cumulative Time



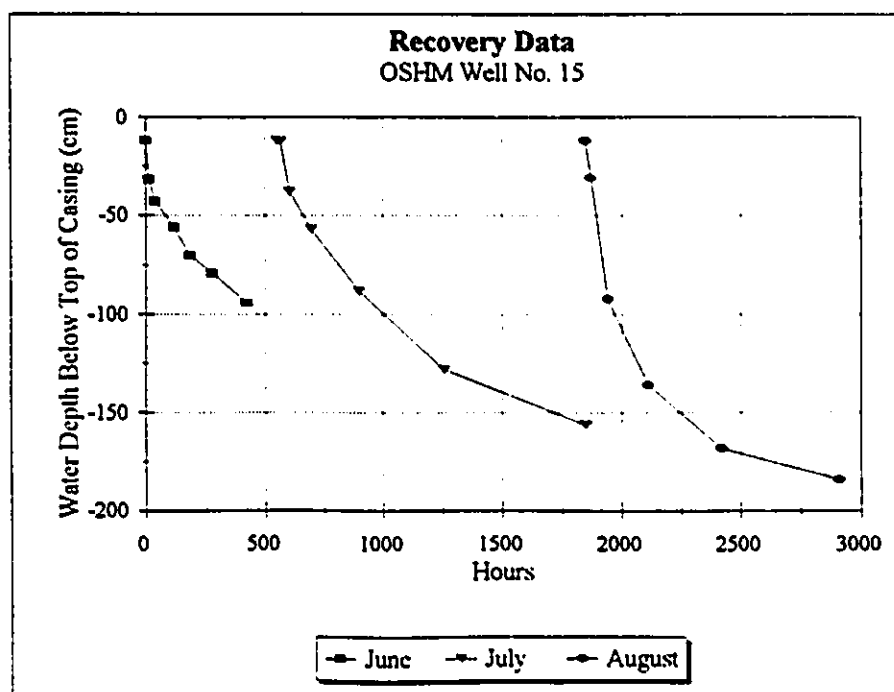
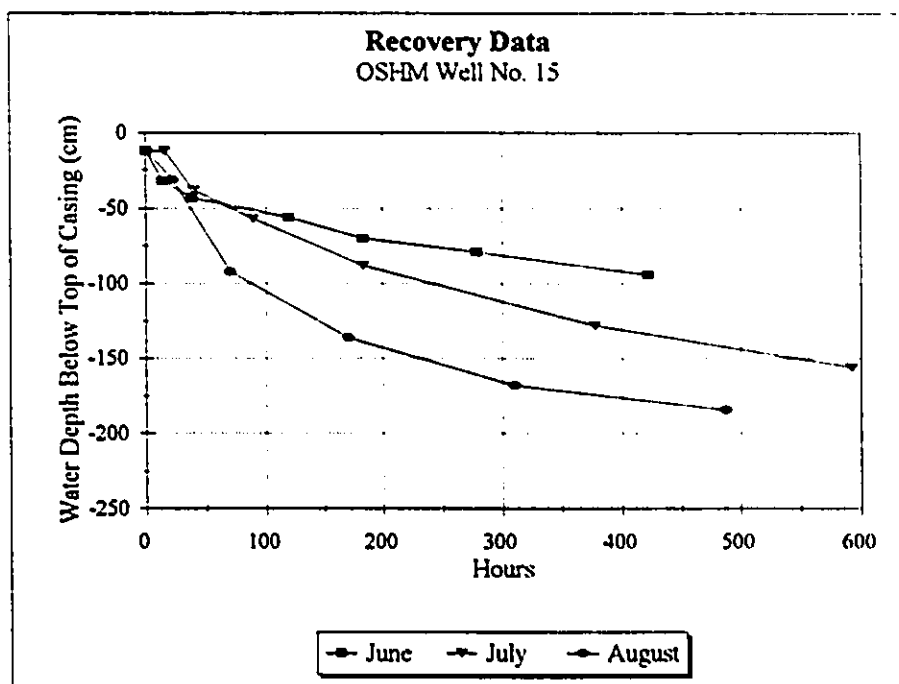
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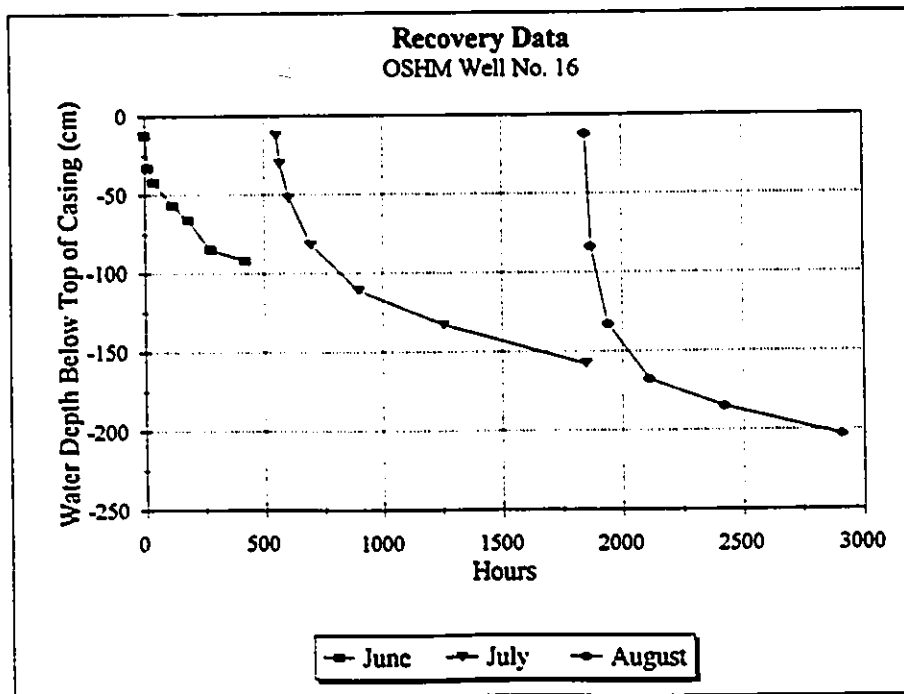
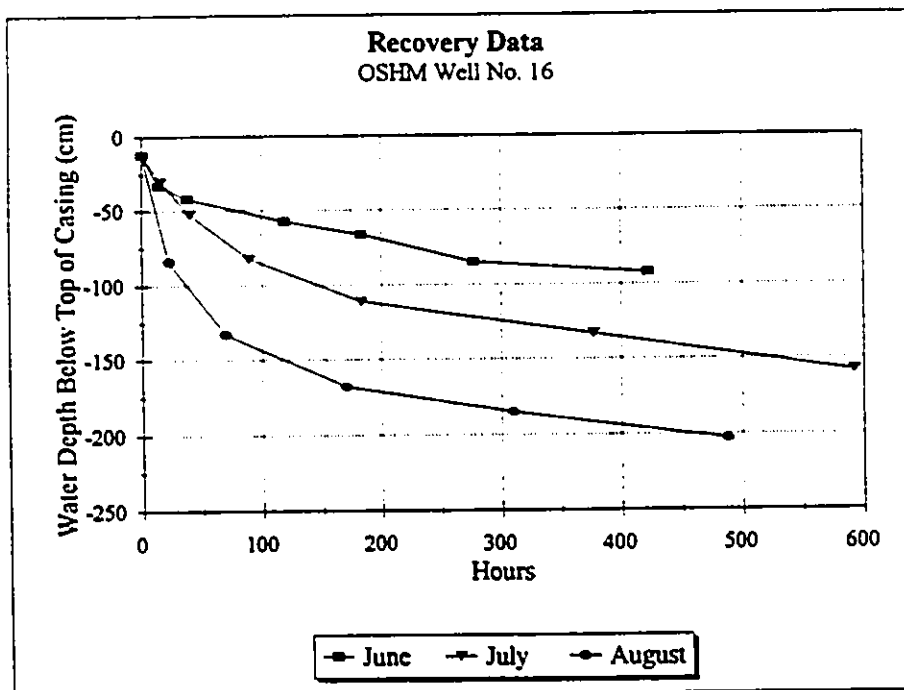
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Individual Tests and Cumulative Time



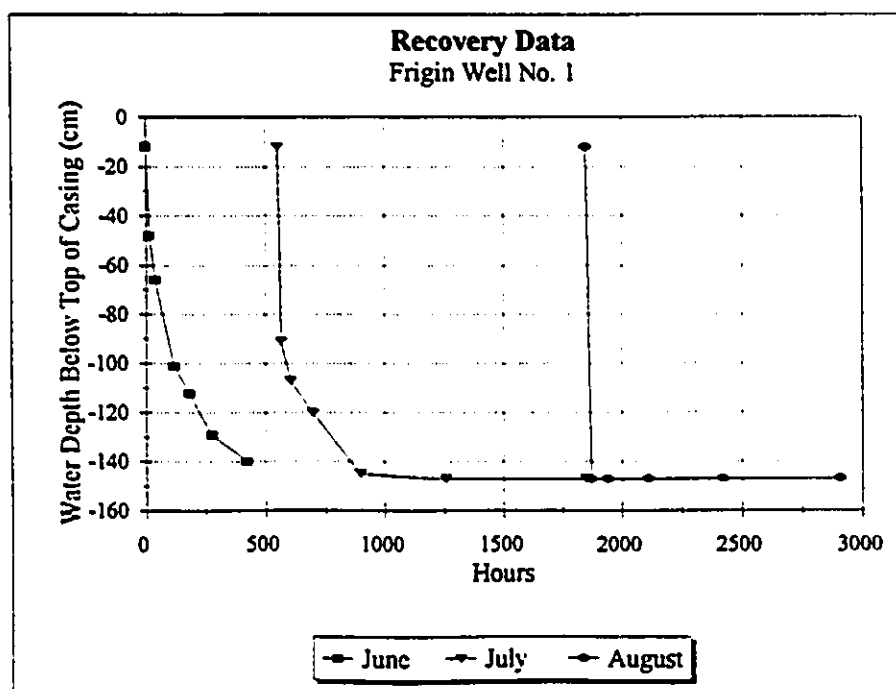
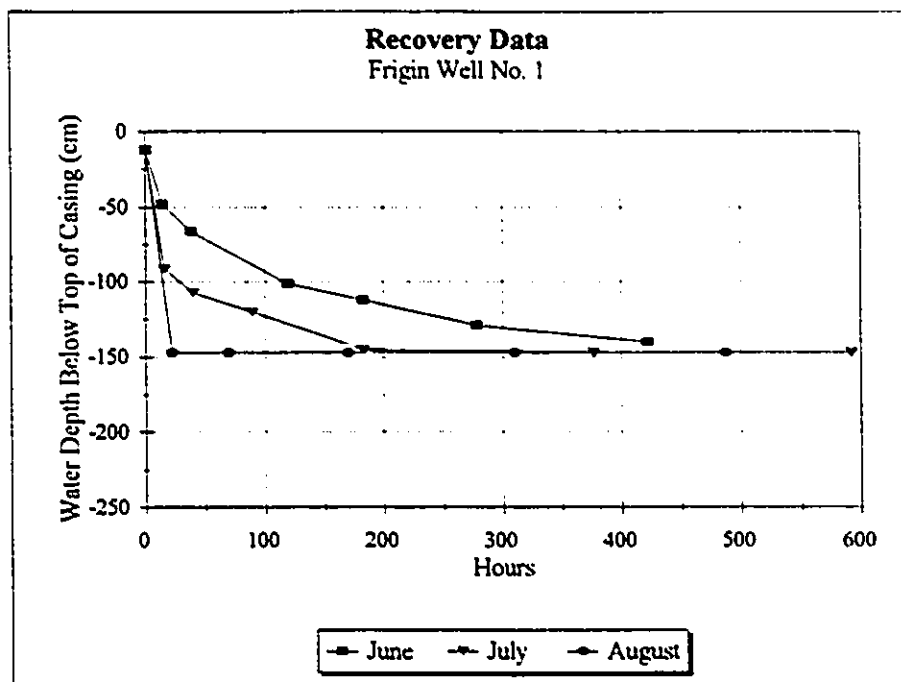
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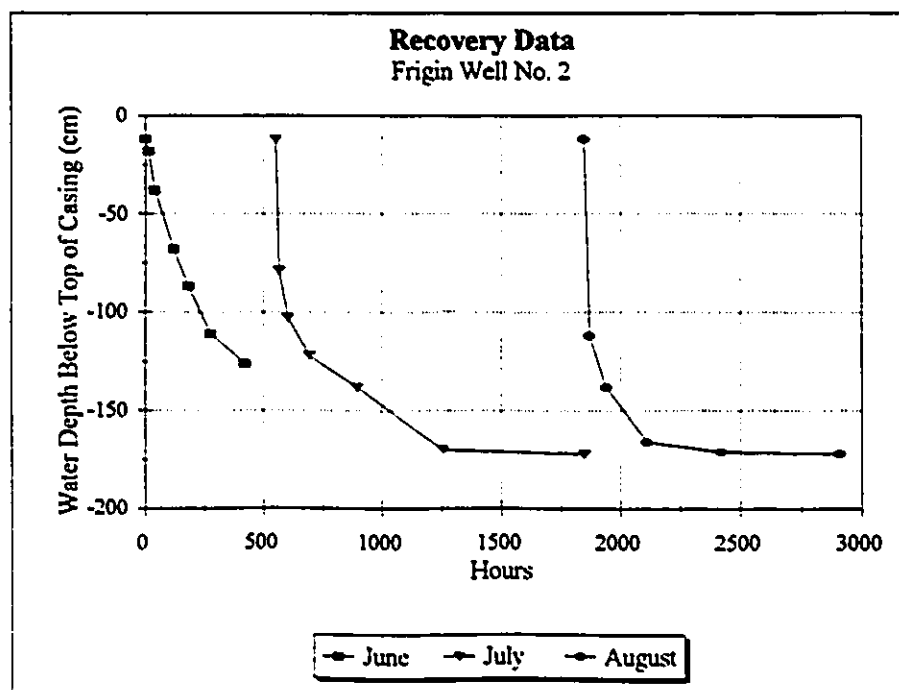
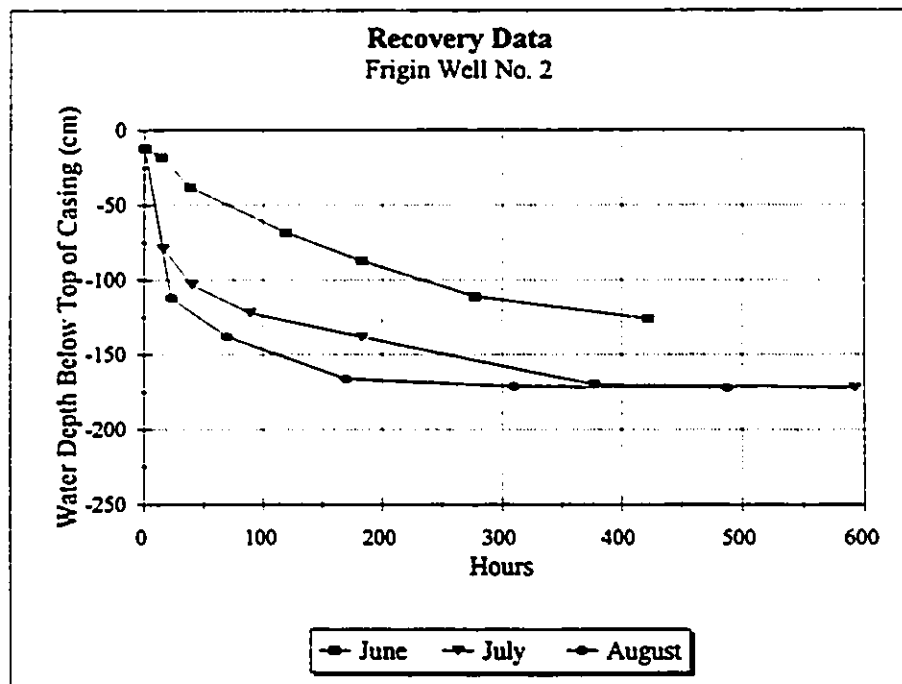
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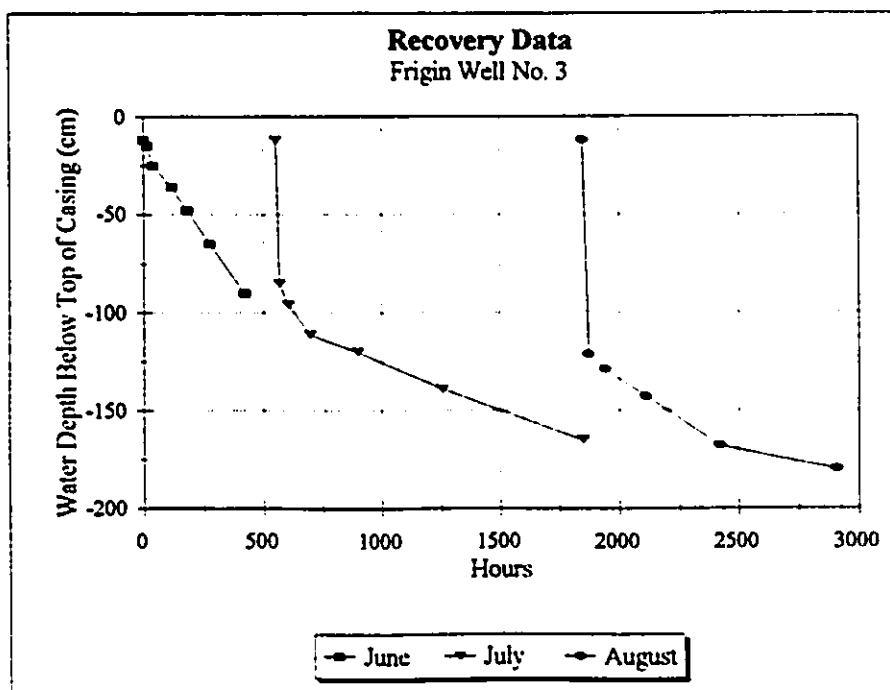
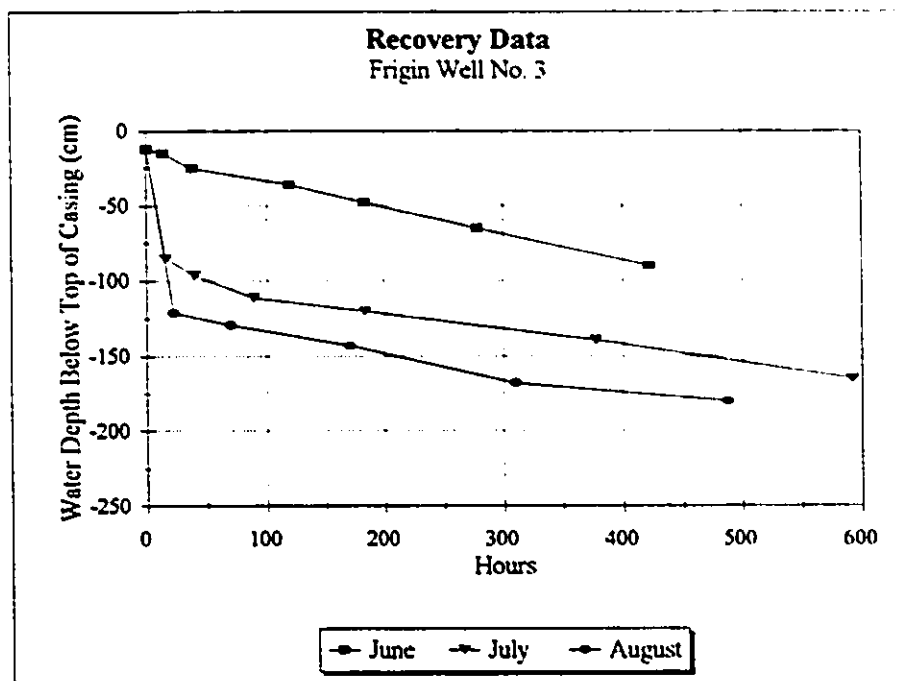
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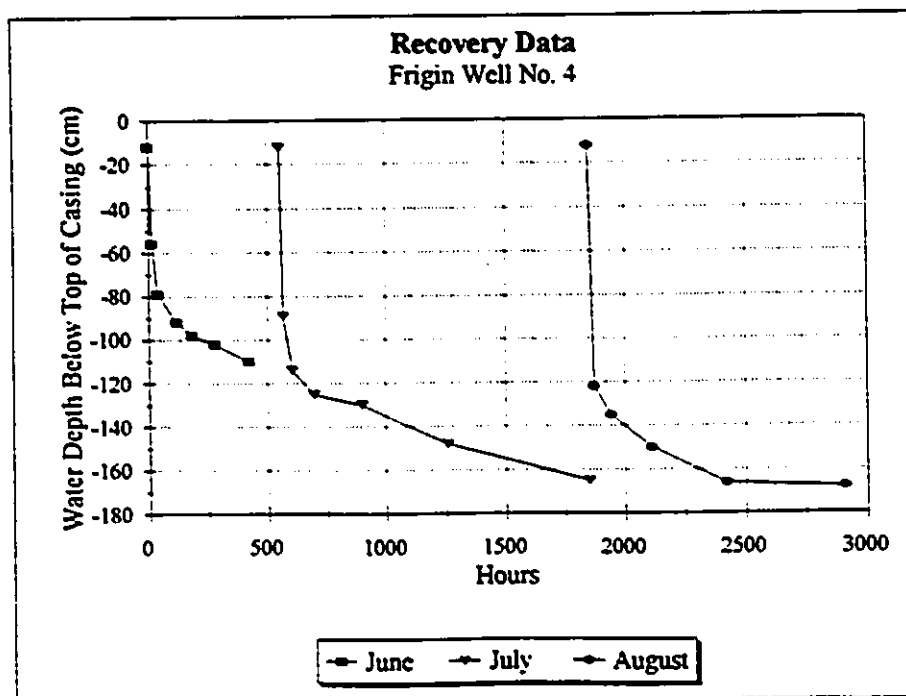
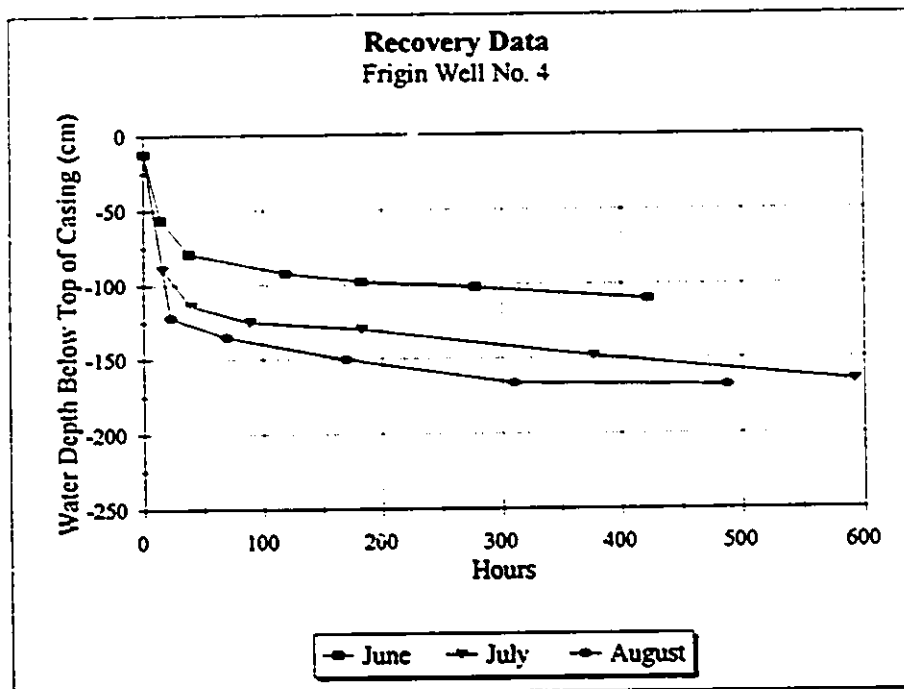
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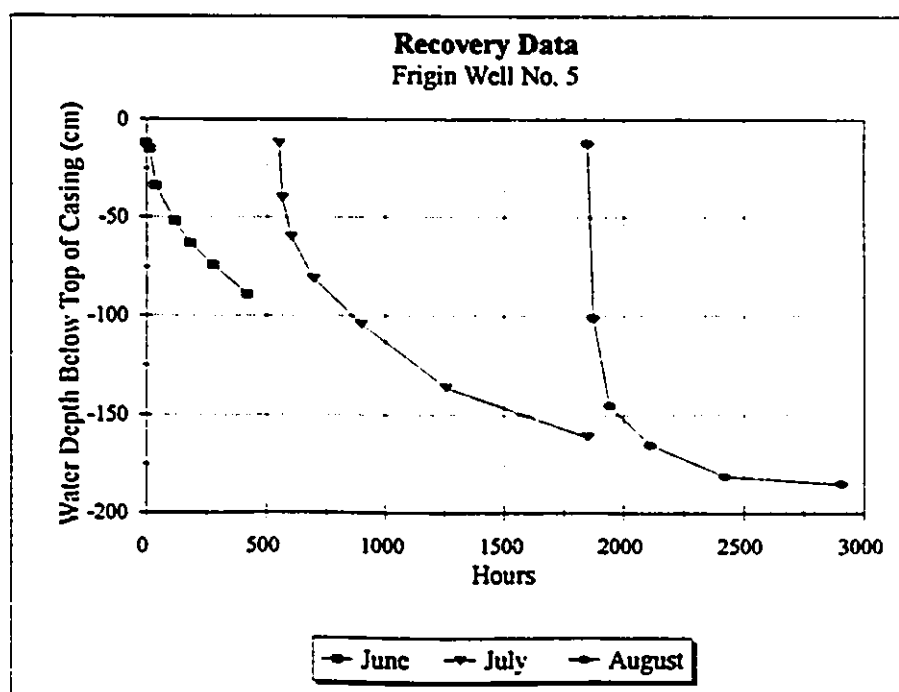
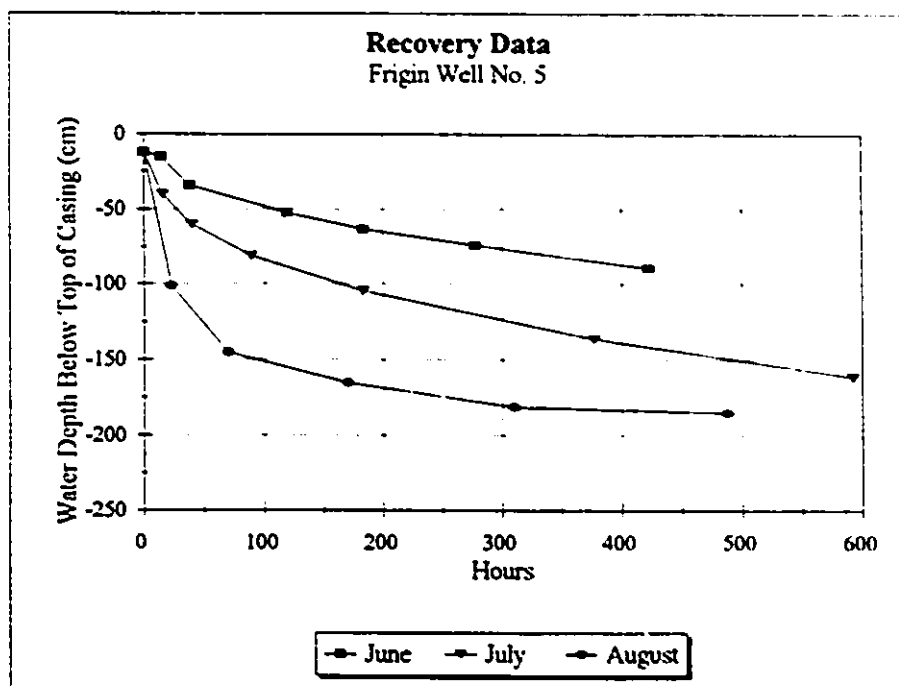
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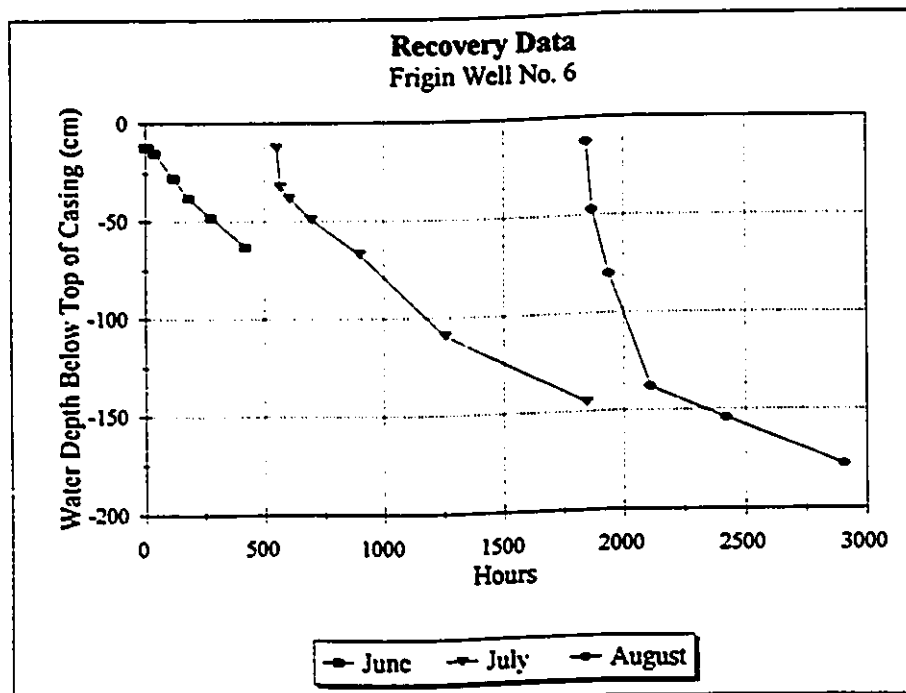
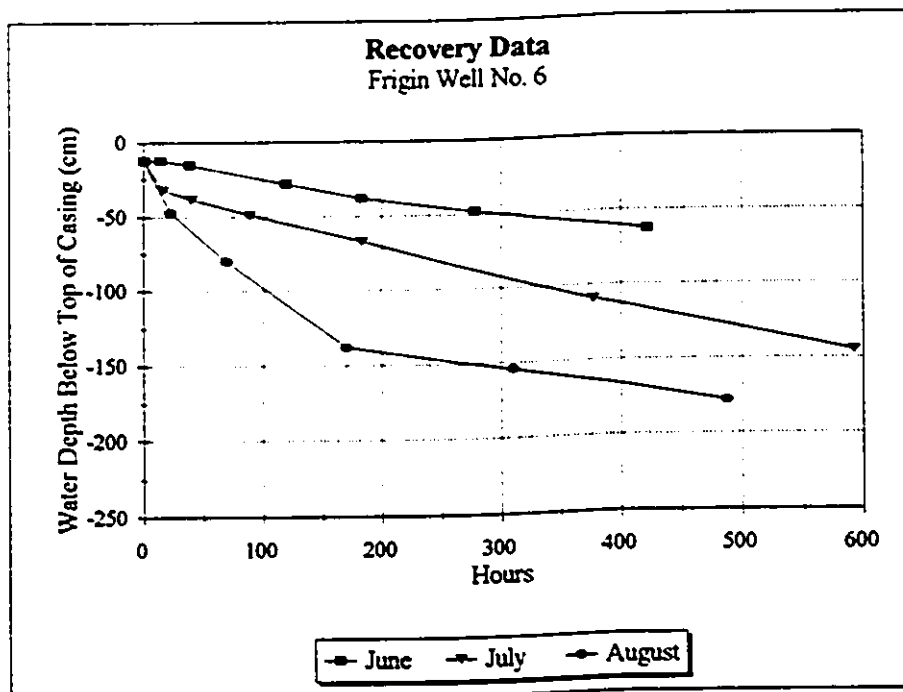
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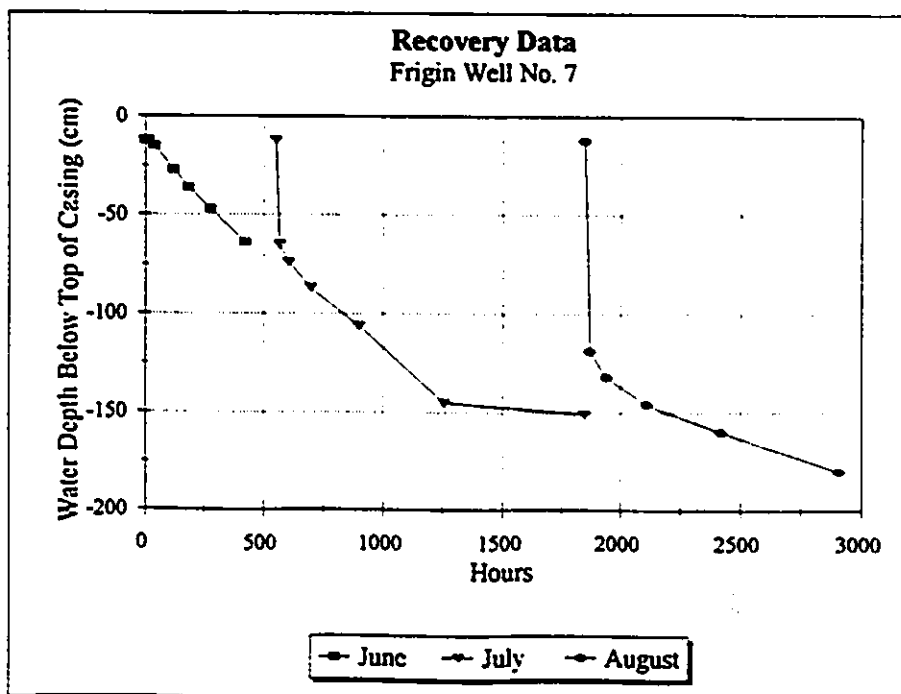
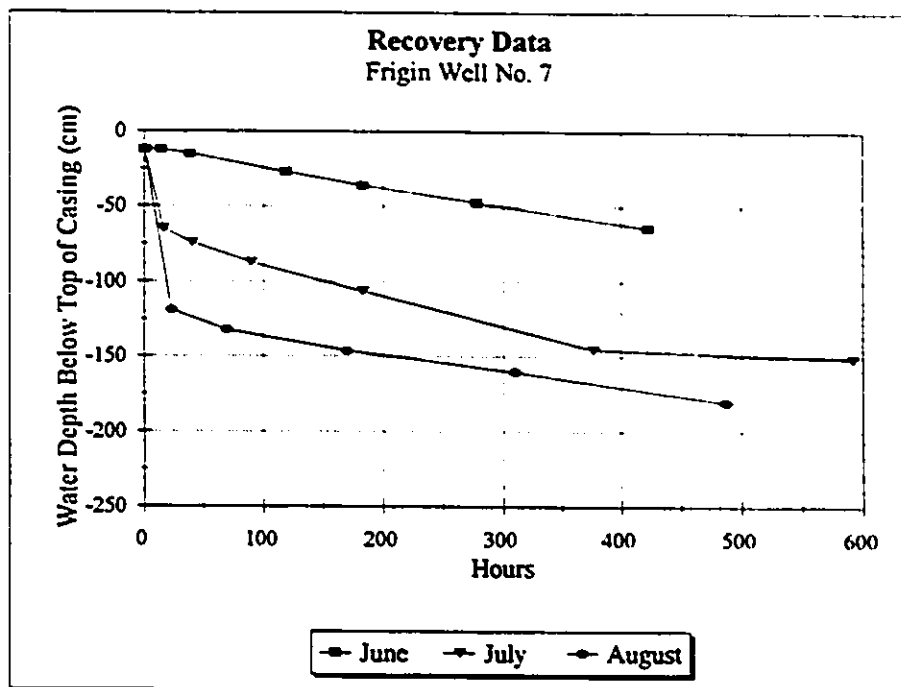
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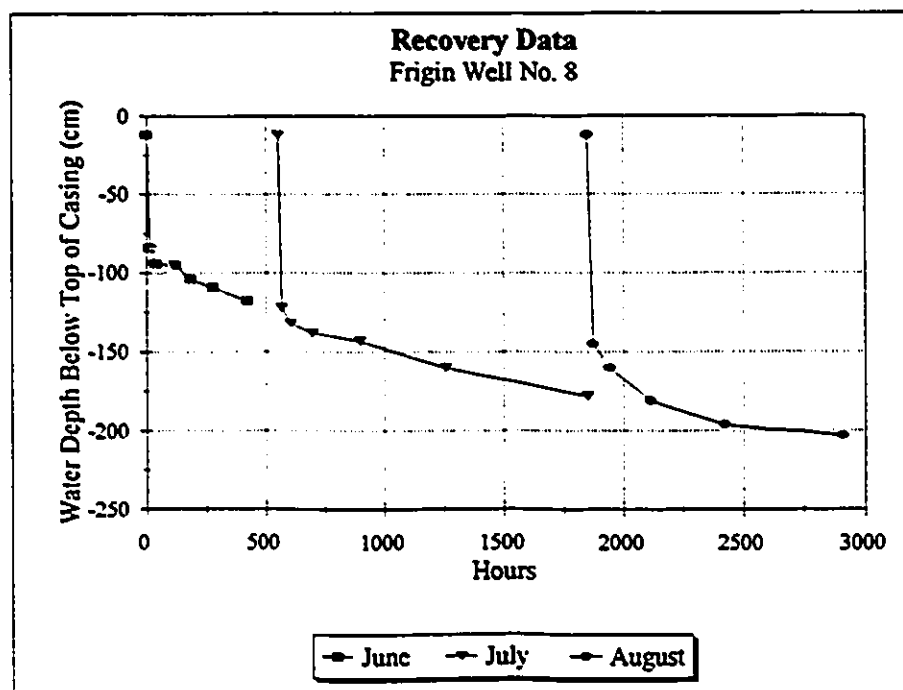
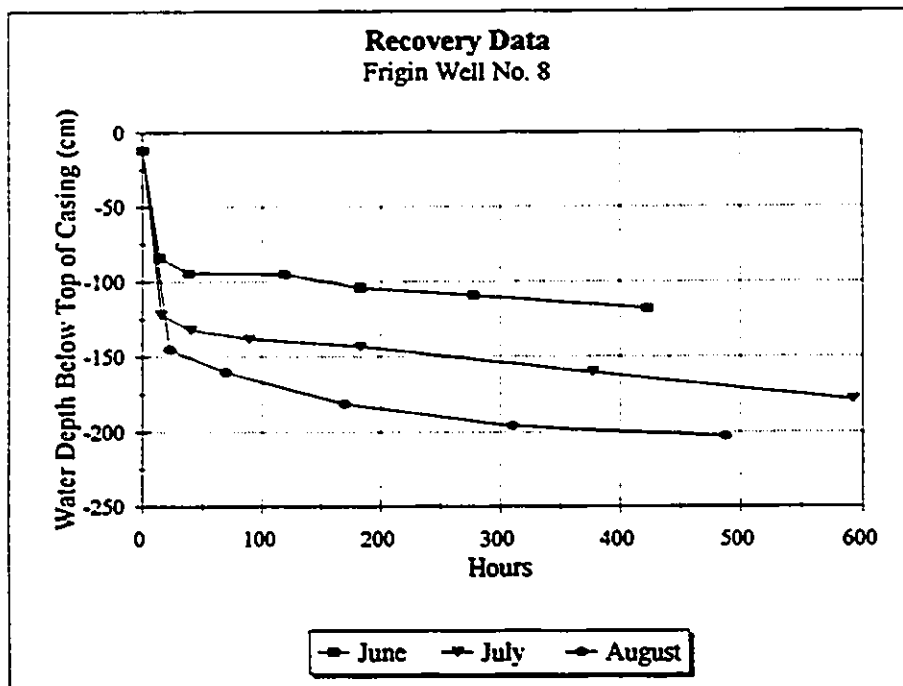
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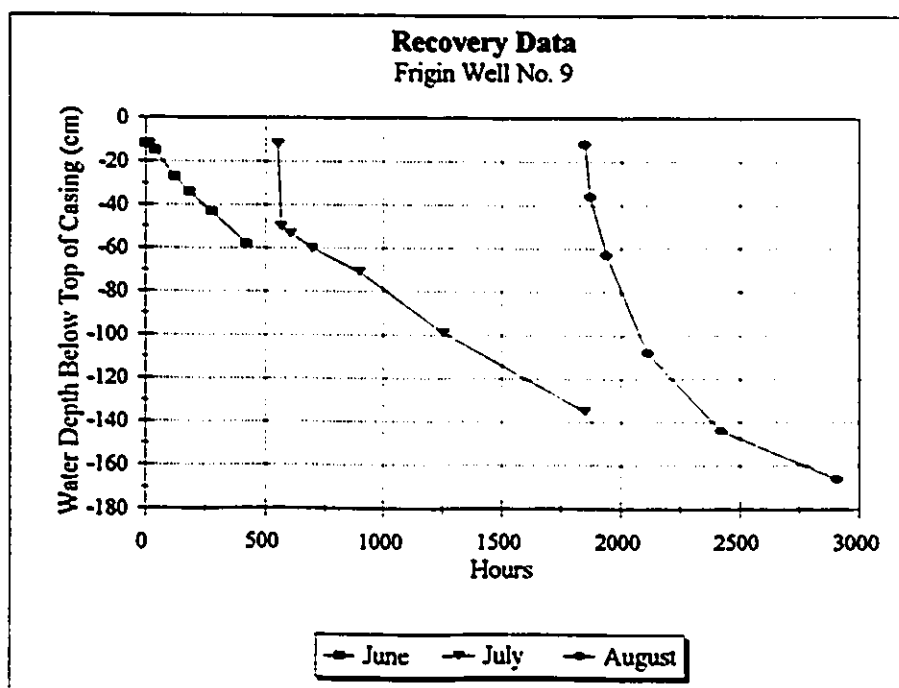
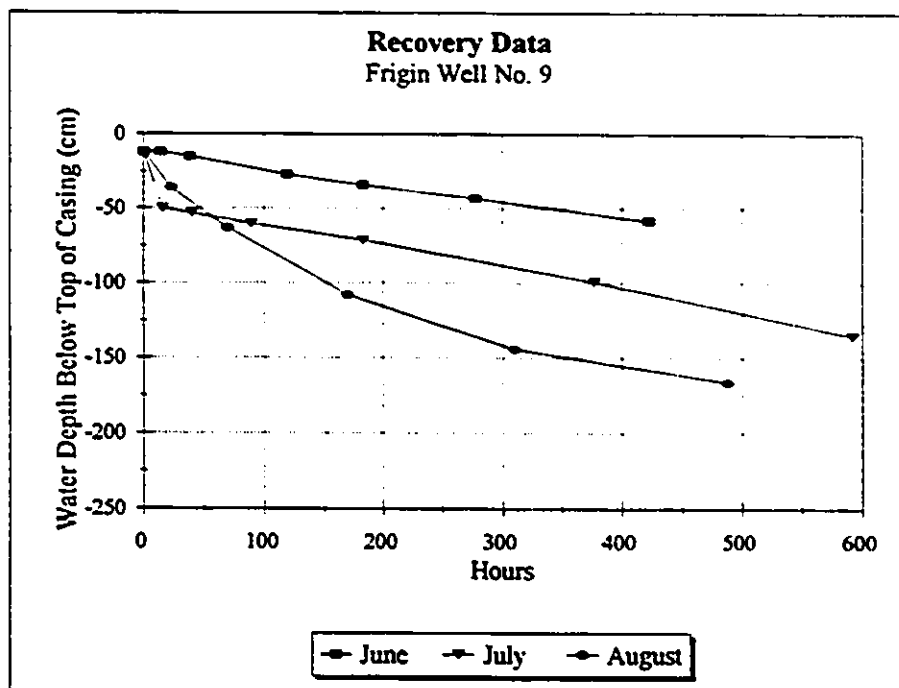
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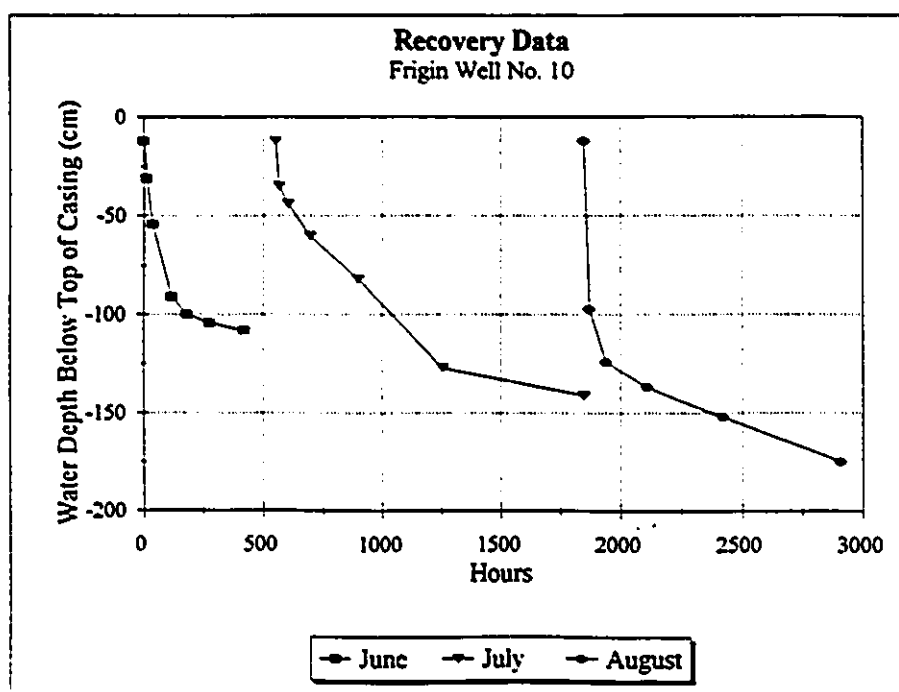
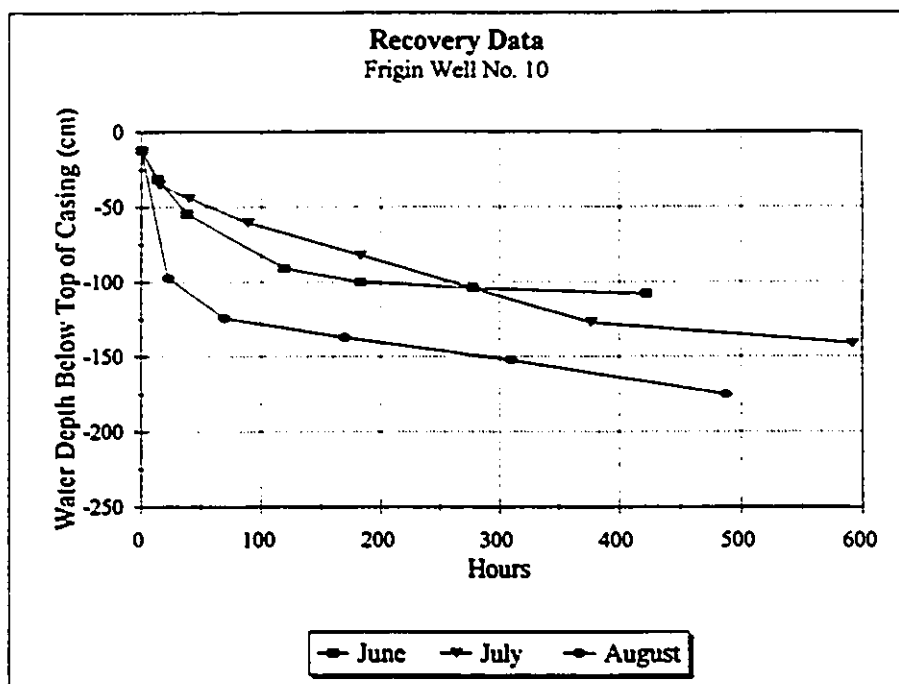
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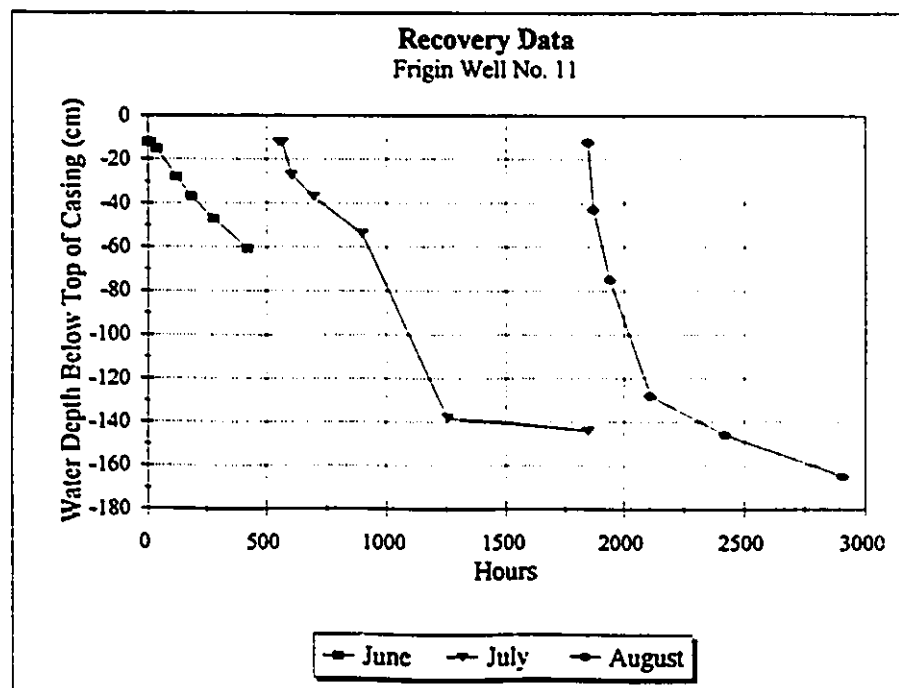
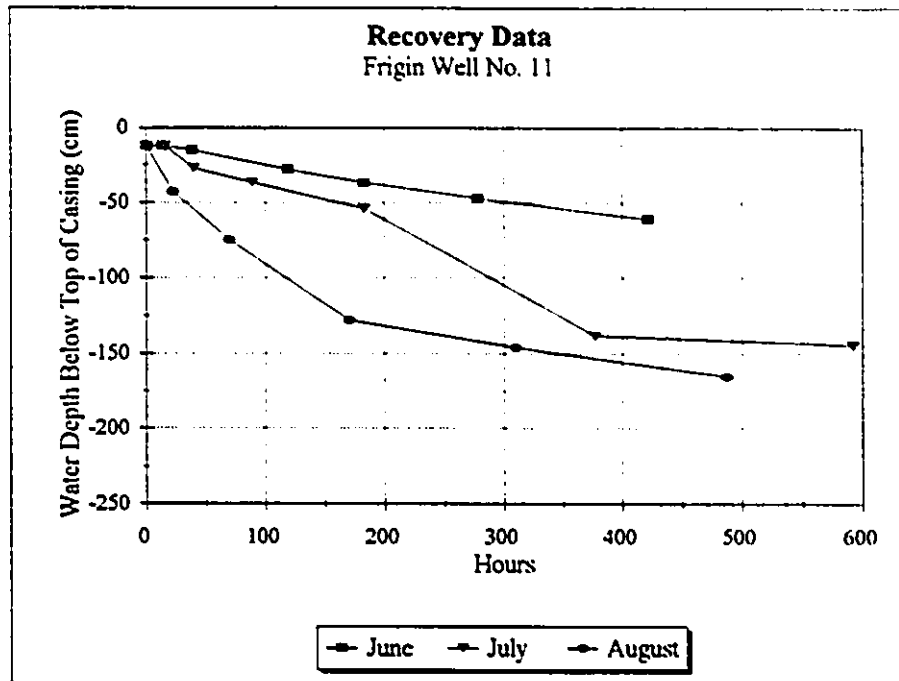
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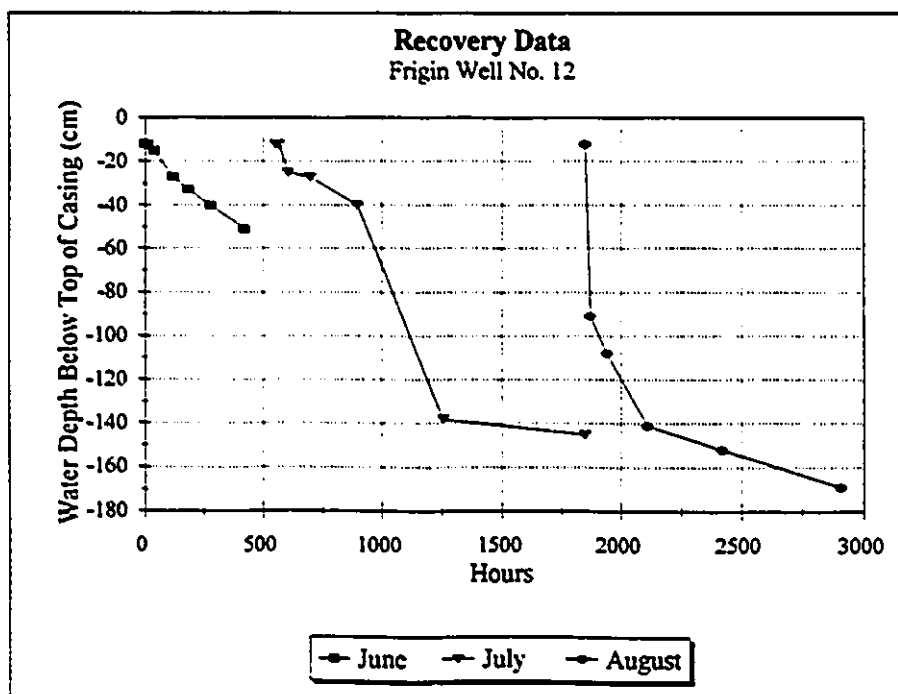
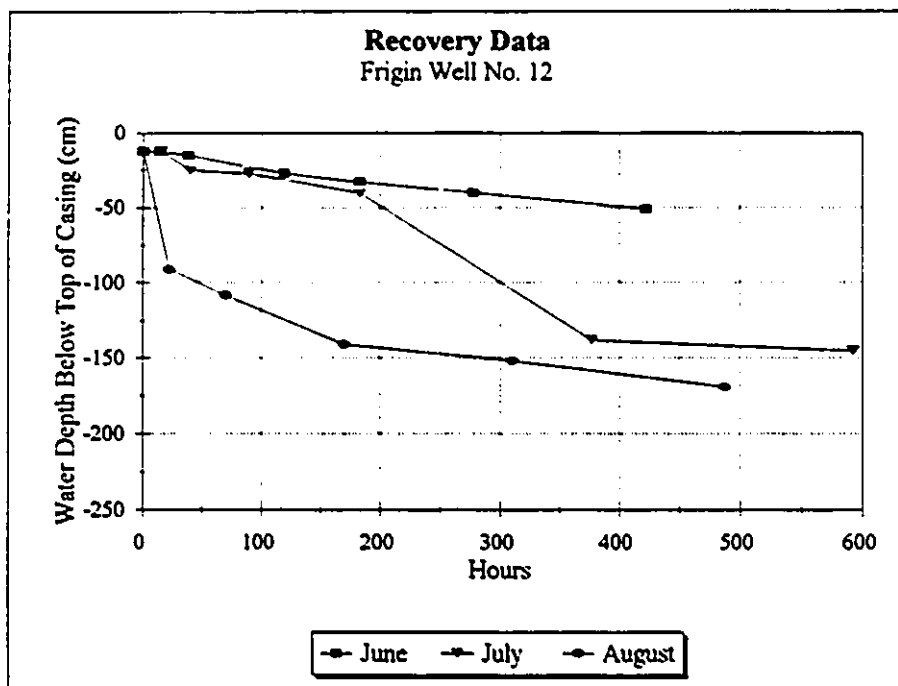
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Individual Tests and Cumulative Time



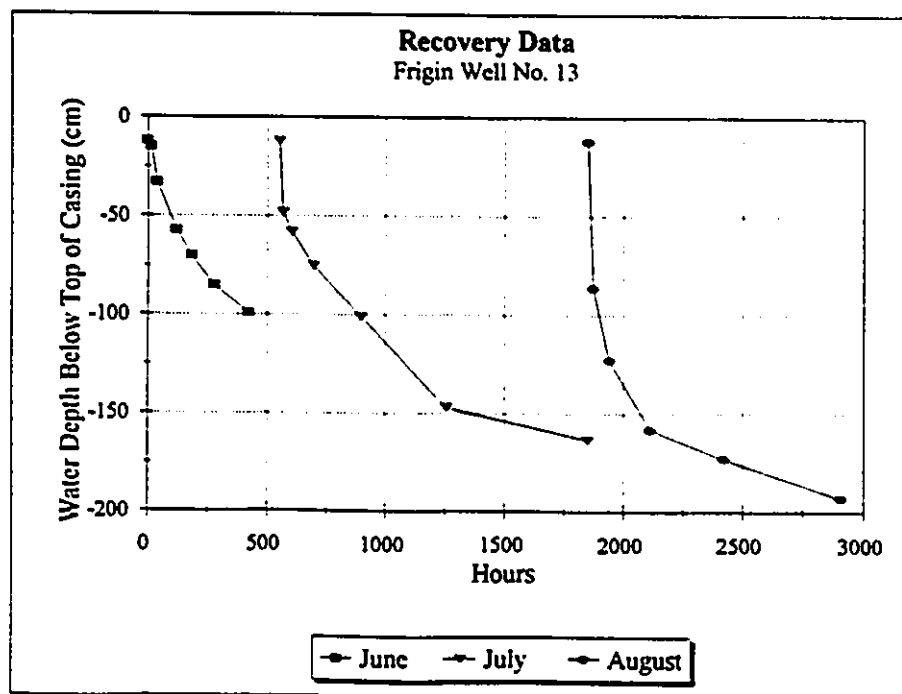
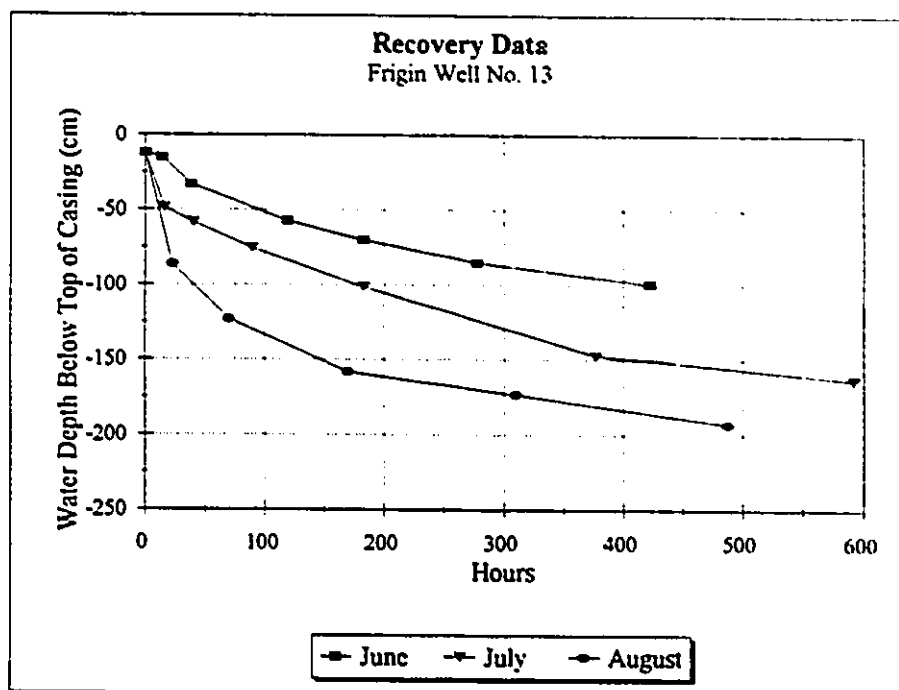
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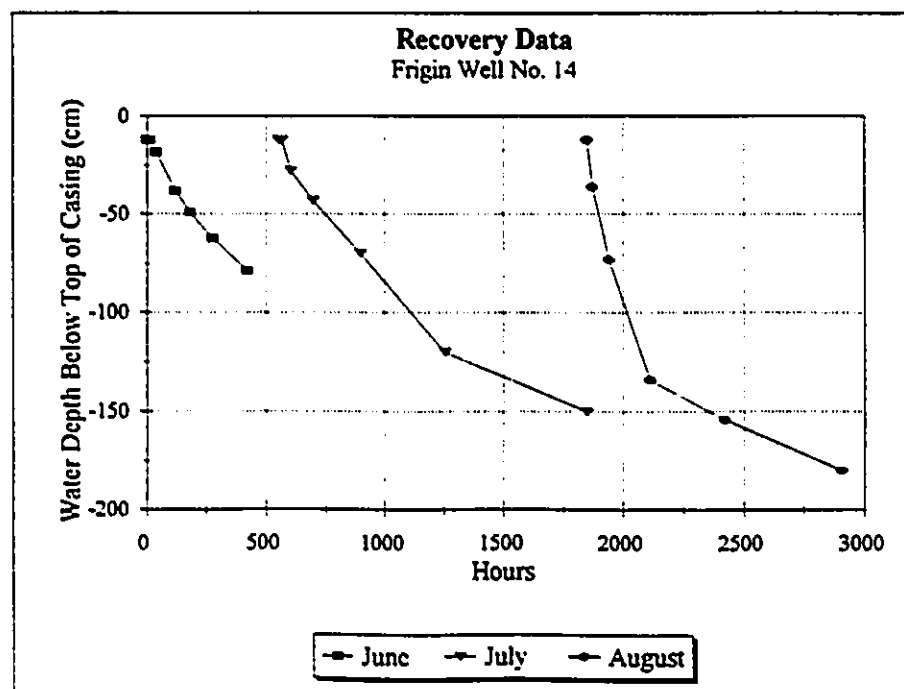
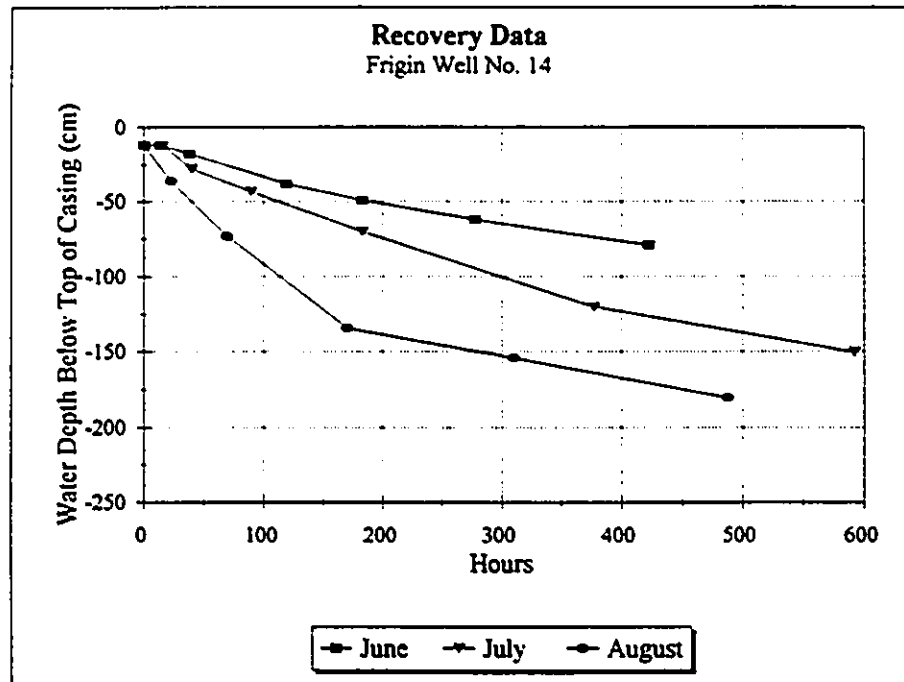
Recovery Data Plots

Individual Tests and Cumulative Time



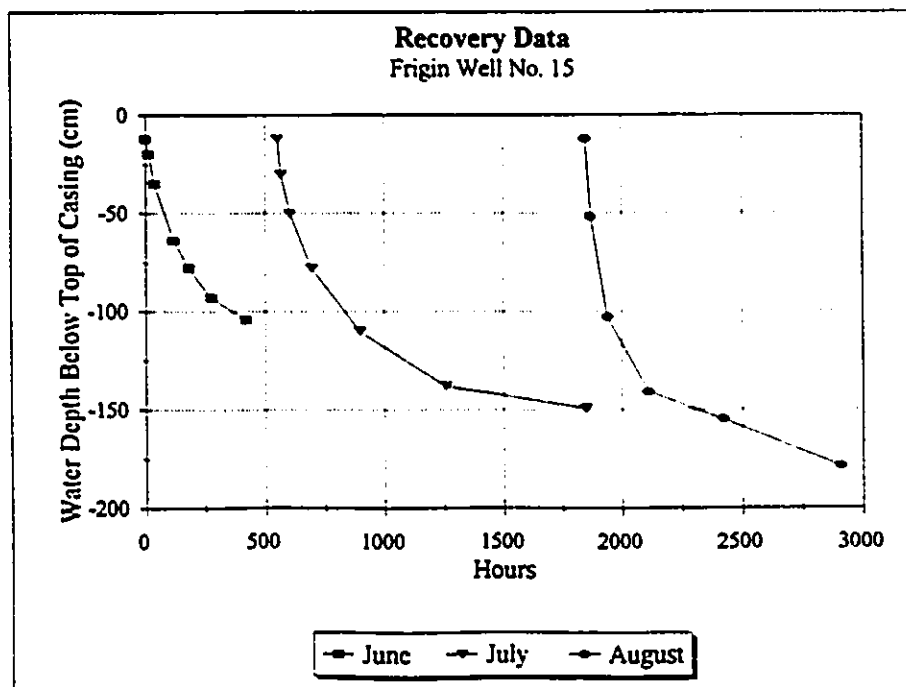
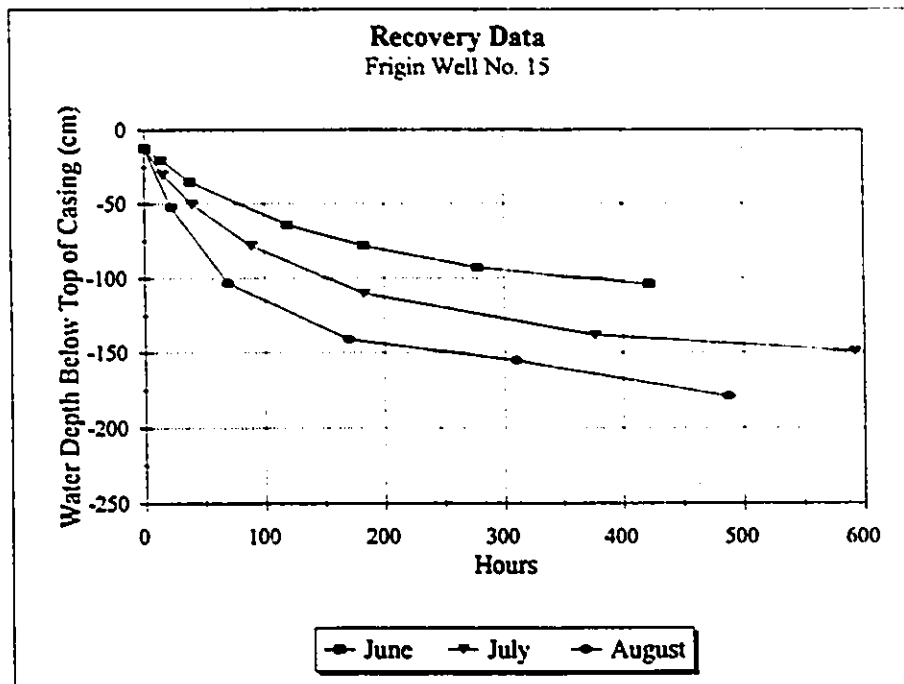
Recovery Data Plots

Individual Tests and Cumulative Time



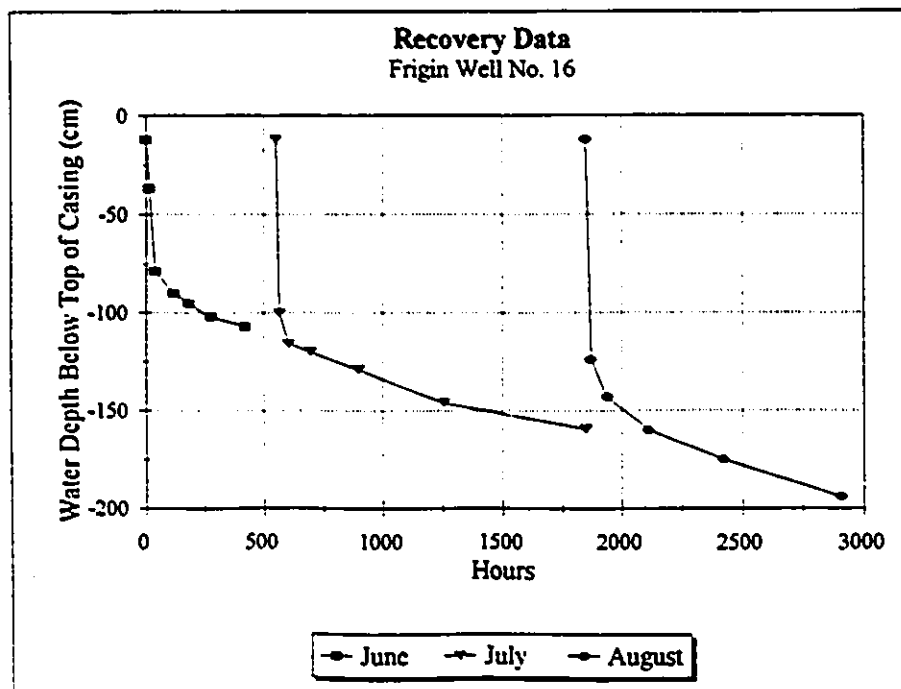
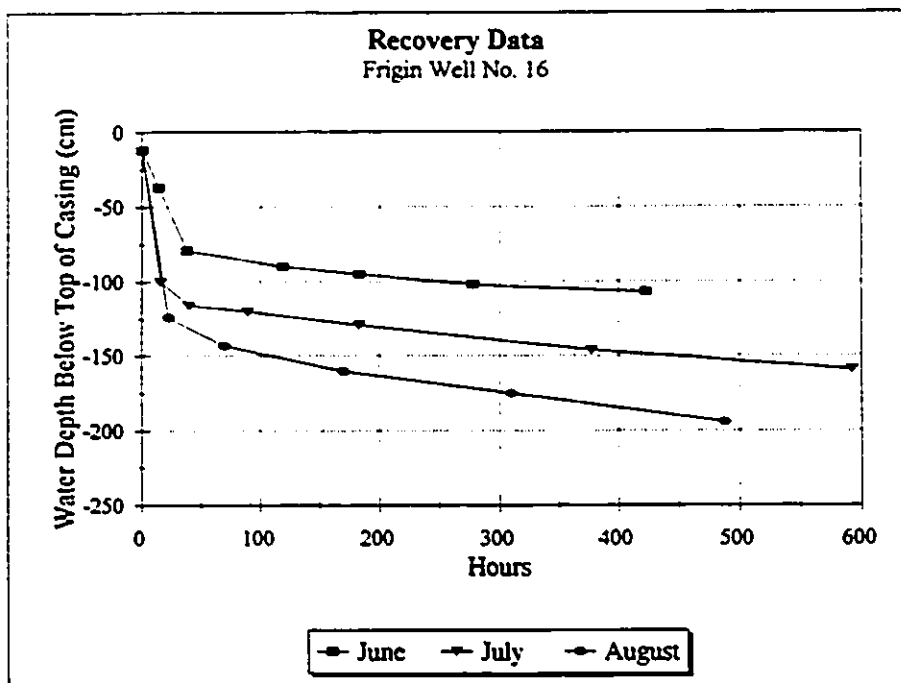
Recovery Data Plots

Individual Tests and Cumulative Time



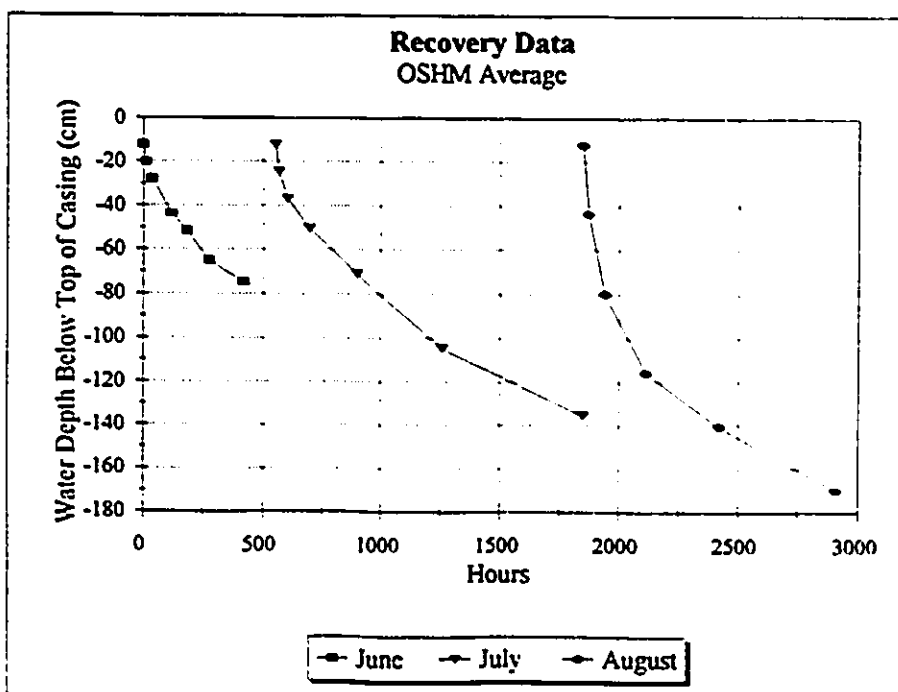
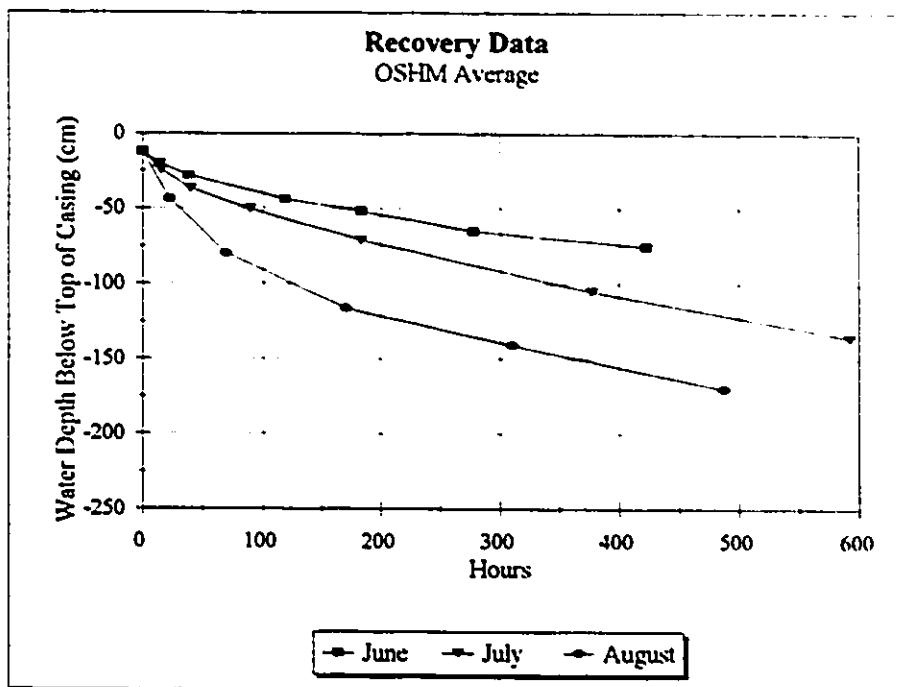
Recovery Data Plots

Individual Tests and Cumulative Time



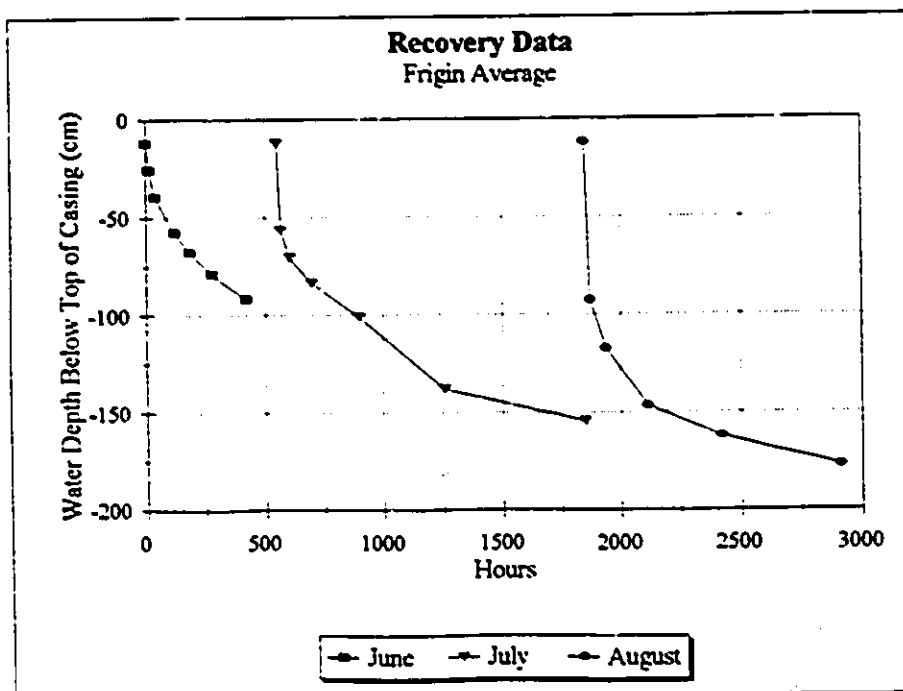
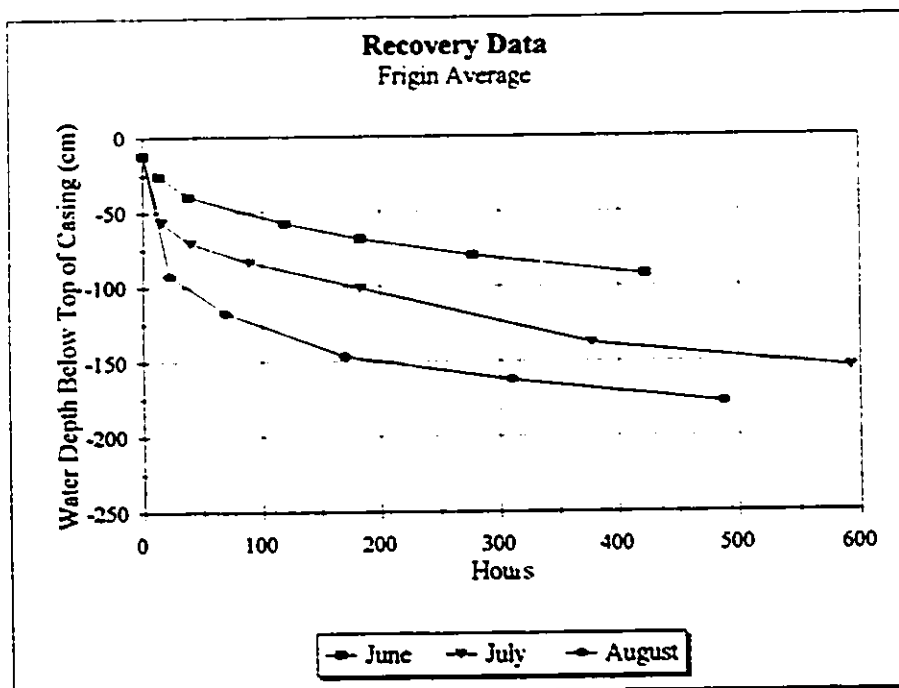
Recovery Data Plots

Individual Tests and Cumulative Time



Recovery Data Plots

Individual Tests and Cumulative Time



APPENDIX C
RECOVERY DATA

Recovery Data for Frigin Piezometers
Piezometers placed Feb. 2, 1993.
All values in cm.
D. to W.F.S. = Depth to Water From ground Surface

Piezometer I.D.	Depth of Piezometer	D.to.W.F.S Feb. 16	D.to.W.F.S Feb. 28	D.to.W.F.S Mar. 7	D.to.W.F.S Mar. 14	D.to.W.F.S Mar. 28	D.to.W.F.S April 8	D.to.W.F.S April 27
F1	147.0	126.5	112.0	82.0	67.0	64.0	37.0	39.0
F2	179.0	152.0	142.0	111.0	88.0	94.0	73.0	44.0
F3	164.5	169.0	159.0	155.0	148.0	90.0	74.0	49.0
F4	165.0	157.0	119.0	117.0	107.0	72.0	41.0	38.0
F5	183.0	137.0	116.0	108.0	94.0	71.0	43.0	40.0
F6	175.5	178.0	164.0	160.0	154.0	138.0	45.0	41.0
F7	190.0	174.0	164.0	106.0	98.0	82.0	58.0	49.0
F8	200.0	81.0	88.0	88.0	76.0	55.0	52.0	40.0
F9	171.5	165.0	150.0	145.0	137.0	127.0	89.0	60.0
F10	170.0	159.0	149.0	145.0	137.0	117.0	68.0	49.0
F11	179.5	183.0	168.0	165.0	157.0	141.0	96.0	58.0
F12	198.0	193.0	177.0	171.0	158.0	130.0	100.0	75.0
F13	184.5	170.0	143.0	139.0	99.0	68.0	47.0	45.0
F14	163.5	167.0	159.0	155.0	149.0	140.0	133.0	113.0
F15	178.5	134.0	118.0	97.0	82.0	51.0	45.0	44.0
F16	203.0	72.0	83.0	83.0	66.0	45.0	59.0	52.0

Recovery Data for OSHM Piezometers

Piezometers placed March 1, 1993.

All values in cm.

D. to W.F.S. = Depth to Water From ground Surface

Piezometer I.D.	Depth of Piezometer	D.to W.F.S. March 28	D.to W.F.S. April 8	D.to W.F.S. April 27	D.to W.F.S. May 2	D.to W.F.S. May 10	D.to W.F.S. May 17	D.to W.F.S. May 27
R1	205.0	172.0	142.0	90.0	84.0	73.0	71.0	78.0
R2	205.0	169.0	152.0	97.0	86.0	70.0	66.0	73.0
R3	205.0	193.0	166.0	115.0	106.0	92.0	86.0	87.0
R4	205.0	193.0	169.0	144.0	133.0	111.0	99.0	90.0
R5	205.0	37.0	47.0	36.0	41.0	46.0	61.0	75.0
R6	205.0	182.0	156.0	102.0	93.0	79.0	73.0	73.0
R7	205.0	186.0	163.0	119.0	108.0	90.0	86.0	79.0
R8	205.0	94.0	75.0	48.0	49.0	46.0	53.0	70.0
R9	205.0	193.0	171.0	144.0	133.0	115.0	105.0	96.0
R10	205.0	185.0	163.0	112.0	103.0	88.0	80.0	77.0
R11	203.0	53.0	47.0	39.0	50.0	48.0	63.0	75.0
R12	205.0	184.0	162.0	104.0	94.0	80.0	74.0	71.0
R13	205.0	98.0	53.0	43.0	46.0	44.0	52.0	70.0
R14	205.0	188.0	167.0	123.0	113.0	97.0	90.0	86.0
R15	205.0	52.0	52.0	48.0	53.0	50.0	61.0	83.0
R16	205.0	151.0	70.0	55.0	53.0	46.0	48.0	69.0

OSHM recovery data for the June slug test.
 All Values in cm.
 Values are depth below top of casing

Piezometer #	June 16 Initial	June 16 Hour 0	June 17 14 hours	June 18 38.5 hours	June 21 119.5 hours	June 24 183 hours	June 28 278 hours	July 4 422 hours
R1	103	12	12	18	42	52	67	87
R2	94	12	12	18	38	48	80	76
R3	96	12	12	15	43	50	81	75
R4	91	12	12	15	25	29	37	49
R5	81	12	39	76	86	94	102	108
R6	81	12	12	15	30	37	48	63
R7	85	12	12	15	31	39	49	62
R8	76	12	12	30	52	63	77	90
R9	94	12	12	12	27	31	39	51
R10	81	12	12	15	27	32	41	52
R11	77	12	66	78	86	91	98	103
R12	78	12	12	15	25	31	40	51
R13	84	12	18	28	47	59	73	88
R14	96	12	12	12	27	34	44	57
R15	91	12	32	43	56	70	79	94
R16	88	12	33	42	57	66	85	92

Frigin recovery data for the June slug test.
 All Values in cm.
 Values are depth below top of casing

Piezometer #	June 16 Initial	June 16 Hour 0	June 16 14 hours	June 17 38.5 hours	June 18 119.5 hours	June 21 183 hours	June 24 278 hours	June 28 422 hours	July 4
F1	105	12	48	66	101	112	129	140	
F2	104	12	18	38	68	87	111	126	
F3	73	12	15	25	36	48	65	90	
F4	97	12	56	79	92	98	102	110	
F5	72	12	15	34	52	63	74	89	
F6	74	12	12	15	28	38	48	63	
F7	67	12	12	15	27	36	47	64	
F8	79	12	84	94	95	104	109	118	
F9	62	12	12	15	27	34	43	52	
F10	80	12	31	54	91	100	104	108	
F11	65	12	12	15	28	37	47	61	
F12	62	12	12	15	27	33	40	51	
F13	87	12	15	33	57	70	85	99	
F14	81	12	12	18	38	49	62	79	
F15	88	12	20	35	64	78	93	104	
F16	85	12	37	79	90	95	102	107	

OSHM recovery data for the July slug test.
All Values in cm.
Values are depth below top of casing

Piezometer #	July 9 Initial	July 9 Hour 0	July 10 15.5 hours	July 11 40 hours	July 13 89.5 hours	July 17 183 hours	July 25 377 hours	Aug. 3 592 hours
R1	104	12	20	31	44	70	134	202
R2	90	12	20	27	40	79	132	180
R3	87	12	30	32	42	58	90	121
R4	60	12	12	12	18	35	62	89
R5	119	12	34	90	101	126	144	158
R6	74	12	12	12	26	44	78	111
R7	75	12	12	12	27	44	76	107
R8	103	12	35	62	94	120	145	165
R9	60	12	12	12	18	34	60	85
R10	62	12	12	20	25	39	64	89
R11	110	12	106	126	132	137	157	176
R12	60	12	12	12	25	36	60	86
R13	103	12	20	30	48	75	107	153
R14	69	12	12	20	27	42	103	131
R15	106	12	12	38	57	88	128	156
R16	100	12	30	52	82	111	133	158

Frigin recovery data for the July slug test.

All Values in cm.

Values are depth below top of casing

Piezometer #	July 9 Initial	July 9 Hour 0	July 10 15.5 hours	July 11 40 hours	July 13 89.5 hours	July 17 183 hours	July 25 377 hours	Aug. 3 592 hours
F1	147	12	91	107	120	145	147	147
F2	135	12	79	103	122	138	170	172
F3	113	12	85	96	111	120	139	165
F4	122	12	89	114	125	130	148	165
F5	101	12	40	60	81	104	136	161
F6	107	12	32	38	49	67	109	145
F7	125	12	65	74	87	106	145	151
F8	130	12	122	132	138	143	160	178
F9	70	12	50	53	60	71	99	135
F10	120	12	35	44	60	82	127	141
F11	72	12	12	27	37	54	138	144
F12	60	12	12	25	27	40	138	145
F13	124	12	48	58	75	101	147	163
F14	93	12	12	28	43	70	120	150
F15	115	12	30	50	78	110	138	149
F16	118	12	100	116	120	129	146	159

OSHM recovery data for the August slug test.
 All Values in cm.
 Values are depth below top of casing

Piezometer #	Aug. 3 Initial	Aug. 3 Hour 0	Aug. 3 22.5 hours	Aug. 4 70 hours	Aug. 6 170.5 hours	Aug. 10 310 hours	Aug. 16 487.5 hours	Aug. 23
R1	202	12	36	96	198	202	203	203
R2	180	12	63	105	172	197	203	203
R3	121	12	12	47	75	114	169	169
R4	89	12	12	31	54	83	125	125
R5	158	12	76	137	161	177	193	193
R6	111	12	12	36	67	109	174	174
R7	107	12	12	32	62	100	148	148
R8	165	12	71	131	174	187	200	200
R9	85	12	12	30	52	81	132	132
R10	89	12	12	27	48	74	105	105
R11	176	12	151	170	186	193	200	200
R12	86	12	12	27	50	80	120	120
R13	153	12	76	138	174	190	203	203
R14	131	12	27	47	74	109	154	154
R15	156	12	31	92	136	168	184	184
R16	158	12	84	133	168	185	203	203

Frigin recovery data for the August slug test.
 All Values in cm.
 Values are depth below top of casing

Piezometer #	Aug. 3 Initial	Aug. 3 Hour 0	Aug. 4 22.5 hours	Aug. 6 70 hours	Aug. 10 170.5 hours	Aug. 16 310 hours	Aug. 23 487.5 hours
F1	147	12	147	147	147	147	147
F2	172	12	112	138	166	171	172
F3	165	12	121	129	143	168	180
F4	165	12	122	135	150	166	168
F5	161	12	101	145	165	181	185
F6	145	12	47	80	138	154	178
F7	151	12	119	132	146	160	180
F8	178	12	145	160	181	196	203
F9	135	12	36	63	108	144	166
F10	141	12	97	124	137	152	175
F11	144	12	43	75	128	146	165
F12	145	12	91	108	141	152	169
F13	163	12	86	123	158	173	193
F14	150	12	36	73	134	154	180
F15	149	12	52	103	141	155	179
F16	159	12	124	143	160	175	194

APPENDIX D
EXAMPLE CALCULATION OF K

The following is an example calculation of the hydraulic conductivity (K) for Piezometer R7 from the August slug test.

After all the water level data are collected, the K of the piezometer can be calculated. The following is the water level data (in cm) and time (in hours) (from the start of the test).

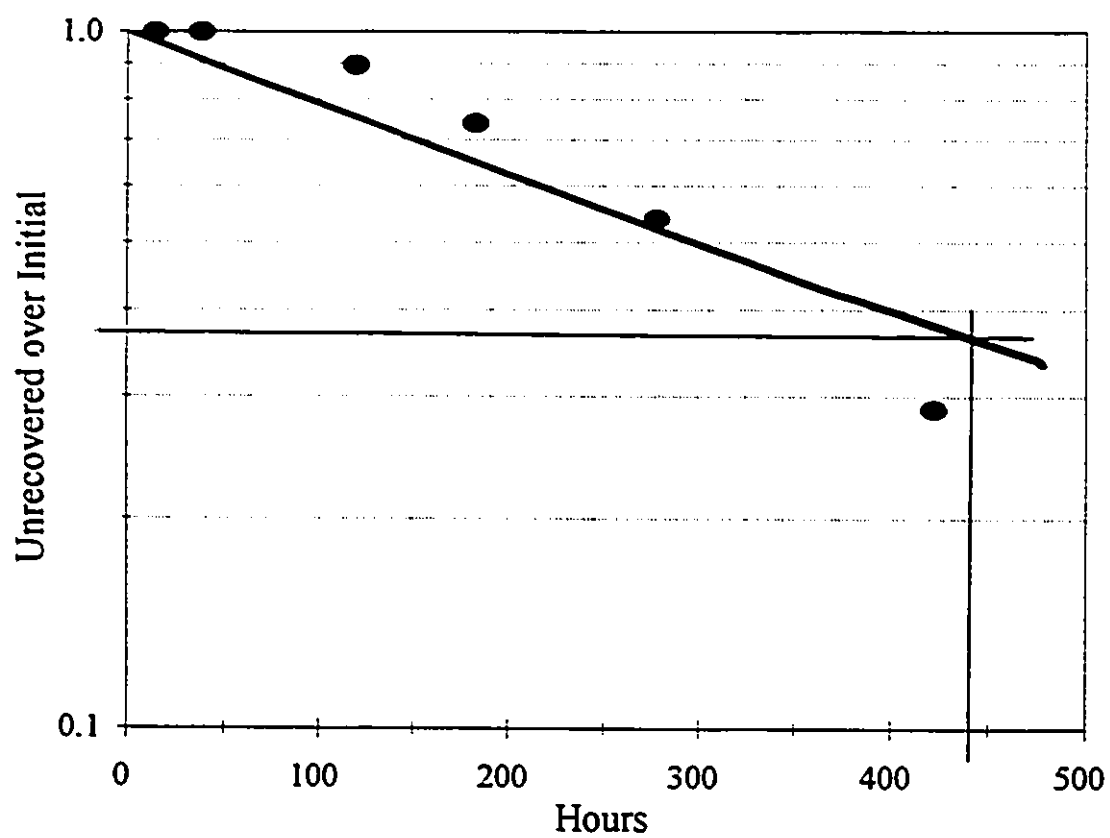
<u>Water Level</u>	0	0	20	50	88	136
<u>Time</u>	0	22.5	70	170.5	310	487.5

From these data, and the static water level (203 cm), the Unrecovered over Initial Head difference can be calculated. For example, at 70 hours, the Unrecovered over Initial Head difference is:

$$(203-20)/(203) = 0.9015 = 0.90.$$

Unrecovered Initial	1.0	1.0	0.90	0.74	0.54	0.29
------------------------	-----	-----	------	------	------	------

The Unrecovered over Initial Head difference is plotted against time on semi-log graph paper. The Unrecovered over Initial Head difference is plotted on the log scale and the time is plotted on the linear scale. A best fit straight line is drawn between the points. The time when this line intersects the 0.37 point from the Unrecovered over Initial Head axis is noted. This time T_0 is used to calculate K. The following page is a plot of the above data.



For this project, the log of the Unrecovered over Initial head values was taken and a linear interpolation was calculated to get the equation of the line. The equation of the line was used to find the time T_0 corresponding to the Unrecovered over Initial head value 0.37. Using this method, T_0 was calculated to be 364 hours. (Graphing on semi-log paper results in a T_0 value of approximately 440 hours. The difference can be attributed to the judgement of the person graphing the line. Linear interpolation was used in this project for consistency.)

Equation 9 in the text, $K = [r^2 \ln(L/R)] / (2 * L * T_0)$, where r is the radius of the piezometer (and sand pack), L is the length of the screen and R is the radius of the screen, is used with T_0 to calculate K . In this case, using T_0 calculated from a graph,

$$K = [(0.0175)^2 \ln(0.5/0.0375)] / (2 * 0.5 * 440) = 1.8 \times 10^{-6} \text{ m/sec.}$$

A value of 1.6×10^{-6} m/sec was calculated using the linearly interpolated data.

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